U. PORTO
FEUP FACULDADE DE ENGENHARIA
UNIVERSIDADE DO PORTO

PERFORMANCE EVALUATION OF EXISTING BUILDINGS IN THE AUGUST 24, 2016 CENTRAL ITALY EARTHQUAKE

MARIA HELENA PAIXÃO COSTA

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Orientador: Professor Doutor Humberto Salazar Amorim Varum

Coorientador: Professor Doutor Guido Camata

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DEPARTAMENTO DE ENGENHARIA CIVIL

Tel. +351-22-508 1901

Fax +351-22-508 1446

✉ miec@fe.up.pt

Editado por

FACULDADE DE ENGENHARIA DA UNIVERSIDADE DO PORTO

Rua Dr. Roberto Frias

4200-465 PORTO

Portugal

Tel. +351-22-508 1400

Fax +351-22-508 1440

✉ feup@fe.up.pt

http://www.fe.up.pt

Aos meus pais

Life is like riding a bicycle. To get your balance you just keep on moving.

Albert Einstein

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- All the colleagues in Asdea for the good welcoming and for all the support.
- My family and friends.

RESUMO

Este trabalho destina-se à avaliação de danos de edifícios após o sismo ocorrido no dia 24 agosto de 2016, no centro de Itália. Amatrice e as suas povoacões vizinhas, região afetada situada no cume de pequenas montanhas, ficou destruída após a ocorrência do sismo. No mês seguinte, ocorreram mais dois fortes sismos, nos dias 26 e 30 de Outubro, mas como grande parte das localidades tinha sido evacuada foi reportada apenas uma fatalidade. Este documento baseia-se em fotografias tiradas no local após a ocorrência do sismo. Apenas uma visita ao local foi efetuada e ocorreu em Norcia no dia 27 de Abril de 2017. Norcia, apesar de não ter sofridos danos maiores durante o sismo de 24 Agosto devido às suas eficientes medidas de melhoramento sísmico décadas atrás, enfrentou graves danos à passagem dos seguintes sismos e são estes danos que aqui são reportados.

O rasto de ruína englobou, a morte de quase 300 pessoas, a destruição de vários edifícios públicos e residenciais, históricos e culturais, e a ocorrência de vários deslizamentos e queda de pedregulhos nas estradas de acesso às vilas, propiciados pela topografia do terreno. A causa do evento está relacionada com uma série de falhas ativas ao longo da cadeia dos Apeninos devido à colisão das placas Euroasiática e Africana. A maioria do edificado era alvenaria não reforçada de dois pisos nos centros históricos e estruturas de betão armado na periferia. Durante o trabalho será discutida também a hipótese de amplificação local.

Este documento apresenta duas fases principais sendo a primeira uma exposição teórica dos mecanismos de falha globais e mecanismos de falha locais para informar o leitor sobre todos os possíveis modos de rotura dos edifícios de alvenaria e dos edifícios de betão armado. A segunda fase é uma particularização da primeira, sendo expostas fotografias de Amatrice, Pescara del Tronto, Arquarta del Tronto e Accumoli e descritos os danos visualizados. Esta fase está dividida em dois capítulos: o primeiro faz um levantamento dos padrões de danos observados e exibe mapas contendo as localizações dos edifícios mais representativos. São também elaboradas tabelas que avaliam o nível de dano sofrido pelos mesmos edifícios com base num esquema utilizado pela proteção civil em caso de emergências pós-sismo. Tentativas de zonamento para Pescara del Tronto e Arquata de Tronto são realizadas. O segundo capítulo aponta possíveis causas para os colapsos e/ou danos severos nas estruturas. Uma comparação antes e depois é também realizada para melhor demonstrar ao leitor o impacto do sismo.

Apesar de esta região de Itália ser uma zona de alto risco sísmico e a memória de semelhantes eventos passados ser extensa, ainda são muito poucas estruturas melhoradas. Este facto é devido aos altos custos que implicam o referido melhoramento e em muitos casos as casas são apenas casas de verão, ou seja, de habitação secundária. Noutros casos o melhoramento pode não ser compatível com a estrutura original por ser alvenaria antiga, em muitos casos com falta de manutenção.

PALAVRAS-CHAVE: MASONRY, REINFORCED CONCRETE, FAILURE MECHANISMS, VULNERABILITY ASSESSMENT, SEISMIC RETROFIT

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ABSTRACT

This document aims at assessing the post-earthquake building damage occurred on the August 24, 2016, in central Italy. Amatrice and nearby villages, affected region located on top of ridges, was destroyed after the earthquake. In the following month, two more strong earthquakes occurred, October 26 and 30, but as the villages have been previously evacuated only one death was reported. This work is based on photos locally taken after the earthquake. Only one visit was done and occurred in Norcia on April 27, 2017. Norcia although did not suffer major damage during the August 24, due to its effective retrofit measures implemented decades before, faced serious damage in October and these are the ones here reported.

The track of ruin encompassed the death of almost 300 people, the destruction of several residential, public and historical/cultural buildings, and the occurrence of many landslides and rock falls on the roads accessing the villages, propitiated by the topography of the soils. The cause of the earthquake is related to a series of active faults along the Apennines chain due to the collision of the African and Euroasian plates. Most of the built in the region was unreinforced old masonry mainly in historic centres and reinforced concrete structures in the suburbs. During this work, the hypothesis of site amplification will also be discussed.

This document presents two distinct main phases. The first is a theoretical exhibition of the global and local failure mechanisms to inform the reader about all the possible failure modes in both masonry and reinforced concrete buildings. The second phase is a particularization of the first, where photos of Amatrice, Pescara del Tronto, Arquata del Trontoand Accumoli are exposed and the visible damage described. Thisd phase is divided into two chapters: the first makes a data collection on the damage patterns and exhibits maps containing the locations of the damage representative buildings. Tables aiming at assessing the level of damage are also presented. This classification is based on the scheme used by the Civil Protection in cases of post-earthquakes damage reconnaissance. Attempts of damage zonation in Pescara del Tronto and Arquata del Tronto finalize the chapter. The second chapter points out possible causes for the collapses/severe damage of the structures. A comparison between the before and after scenarios closes the chapter. This comparison aims at giving the reader the devastating impact of the earthquake.

Although Amatrice and nearby villages are a high seismic risk zone and the in memory of past events many were devastating, only few structures are retrofitted. This fact is due to the high costs inherent to the referred improvement and also in many cases the houses are summer houses, not a primary residence. In other cases, the improvement may not be compatible with the original building because it is old masonry often with lack of maintenance.

KEYWORDS: MASONRY, REINFORCED CONCRETE, FAILURE MECHANISMS, VULNERABILITY ASSESSMENT, SEISMIC RETROFIT

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INTRODUCTION

1.1. GENERAL

Earthquakes are natural hazards, destructive to life and property. The main cause of death during an earthquake is buildings collapse hence, to reduce the loss of human lives, buildings must be safe. The kind of damage depends on: the structure of the building, its age, materials, location, vicinity to other buildings, non-structural elements and on the duration and intensity of the earthquake.
Throughout the different ages of architectural history, earthquakes have always represented one of the main causes of damage and losses of cultural heritage, both monumental buildings and historic centres. Post-earthquake damage observation is a remarkable source of information to compile damage patterns and it has been proving, over time, the urgent need to improve knowledge of the seismic behaviour of old masonry buildings and improve reliability of the retrofitting techniques. However, as earthquakes have a low return period (low probability), these retrofitting measures were rarely introduced correctly. Italy is one of the European countries, along with Greece and Turkey, with more seismic risk. Italian awareness about the seismic vulnerability of buildings dramatically increased after two recent earthquakes hit the nation: Umbria-Marche earthquake in 1997 and Molise earthquake in 2002. The latter had a special impact in the community because the victims were 27 children and a teacher after the collapse of a primary school. After these event, a new seismic code was adopted (OPCM, 2003) inspired mainly by Eurocode 8, as well as a new seismic zonation, and also a researcher centre on earthquake engineering was founded, RELUIS *(Rete dei Laboratori Universitari di Ingegneria Sismica*). Along the years, many standards and guidelines have been introduced to improve the seismic code such as Technical Rules for Constructions, proclaimed in 2008 (shortly named NTC 2008).
Last year in August 24, another devastating earthquake occurred, this time hitting the area of Amatrice and nearby small villages, located on top of the hills or ridges of the Apennines. Once again, and following the past examples, the villages were destroyed, hundreds of lives were taken and the reconstruction costs were estimated in five billion euros. The building stock in this area consisted in old masonry buildings, rarely retrofitted, in historic centres and reinforced concrete buildings in the periphery, typical of the central Italy. Later in October 26 and 30 two strong earthquakes also struck the same zone. Most of the built was constructed before any modern anti-seismic standards were instituted.

This document will compile a full report on the damage and retrofit measures observed in the most devastated villages which were Amatrice, Pescara del Tronto and Arquata del Tronto after the 24th August. The damage occurred in the following month will not be object of discussion in this thesis.

The damage analysis of Pescara del Tronto and Arquata del Tronto are based on photos taken after the 24th August. Photos from Amatrice are hence, extracted from previous reports elaborated by specialists. Accumoli is also one village affected by the earthquake and the one with less epicentral distance. Pictures from this village also take part of this report but, like Amatrice, were extracted from previous documents. Therefore, it is important to affirm that this document is only grounded on photographic data and not on local visits to the affected villages.

Other village hit by the earthquake was Norcia, which already has a long seismic history. However, due to the previous implementation of retrofit measures in most of the buildings, Norcia suffered minor damage during the August event. Nevertheless, with the following October events the buildings, which were formerly weakened, were not able to exhibit the same good behaviour and many damage were recorded. A local visit to this village was done on 27th April 2017 to visualize and photograph the failures after the three earthquakes.

As this earthquake is very recent, it just occurred last summer, the lack of information was a difficulty along the research, accompanied by the uncertainty of the bibliography which many times referred the need to deepen the information in further reports. This thesis is based mainly on previous reports from the GEER (Geotechnical Extreme Events Reconnaissance Association), INGV (*Instituto Nazionale di Geologia e Vulcanologia)* and RELUIS.

1.2. Objectives

This document aims at collecting and evaluating the buildings performance during the earthquake in the affected area in order to understand the impact of the event in both masonry and reinforced concrete structures.

1.3. Outline and Organization

In order to concretize the aims aforementioned, the thesis is divided into six chapters each of them specialized in one aim.

The first chapter initially summarizes the principal characteristics of the main shock such as its epicentre, magnitude and intensity and compares this event with the October events. Next, damage “grading” maps are exhibited so the reader can have a generalized comprehension of the injury extension on the three villages followed by a brief description of the damage both in masonry and reinforced concrete buildings. Historical and cultural buildings are also approached due to the devastating failures in churches and bell towers. As the villages are all situated in steep terrains, landslides and rock failures were very common, especially in Pescara del Tronto where occurred the largest landslide due to the retaining wall failure. After the damage description on the 24th August, the active faults along the Apennines chain believed to be responsible for the strong earthquake are presented as well as the hazard seismic map of Italy. This map shows that the epicentre is located in the zone with higher seismic risk. The recorded spectrums by the nearest stations also take part of this chapter and to finalize, it is made a brief resume on the seismic history of the region.

The second chapter is the most extensive chapter and it holds the theoretical component of the document. The methodology to present these mechanisms is always the same: present a theoretical explanation of the mechanism and after illustrating with photos from the affected villages. A characterization of the existent masonry is also made and it follows the same methodology as before: first an introduction to the masonry in general in Italy and then a particularization into the masonry used in Amatrice and nearby villages. Finally, a brief presentation of row buildings, which behave differently from isolated buildings, is made and examples from Pescara del Tronto are exhibited, in this case with a negative effect.

The third chapter aims at classifying damage patterns and evaluating buildings performances in the three villages. The classification is done according to the scheme provided by the Department of Civil Protection (DCP). For each village is presented a photographic report of damage representative buildings, followed by a map with their locations. To resume the entire information, a table is exhibited containing damage descriptions and damage levels for each picture. To cease the sub-chapters a tentative of damage zonation is accomplished.

The fourth chapter, in the first part, points out the main causes for the buildings (both masonry and reinforced concrete) to collapse and each sub-chapter is associated with one cause. The methodology followed is the same above mentioned: first a theoretical resume on the cause and after examples from the affected villages. The second part consists in a before-after comparison: images taken from Google Maps 2011 are compared to the correspondent ones after the earthquake and a brief damage description is made. With this approach, the reader can easily understand the extension and severity of the failures.

The fifth chapter collects and characterizes the retrofit measures found in the masonry buildings and illustrates with examples of positive and negative improvements. Finally, the last chapter presents the main conclusions.

Figure 1- Pescara del Tronto

2
AMATRICE 24TH, 2016

2.1. SEISMIC EVENT DESCRIPTION

On 24th August 2016, a devastating earthquake struck the central sector of the Apennines among the Lazio, Umbria, Marche and Abruzzo regions, including six provinces, Perugia, Ascoli, Piceno, Fermo, Rieti, L'Aquila and Teramo, and other seventeen municipalities. The most affected area has a radius of 20 km around the epicentre and includes the town of Amatrice and the nearby villages, Accumoli, Pescara del Tronto and Arquata del Tronto. Earthquake shaking was felt in Rome (120 km SW) and Florence (220km NW).

Figure 2.1- Central Italy: Amatrice , Accumoli, Arquata del Tronto and Pescara del Tronto, Image from NASA/USGS Landsat 8 satellite.

The earthquake hit the Central Italy at 03:36, local time. Considerable damage to buildings were detected in Amatrice and in the surrounding areas in the following hours. The magnitude of the earthquake was M 6.0 and the epicentre was near Accumoli, province of Rieti, almost 15 Km from Amatrice, with an estimated depth of 8 km. One hour later an aftershock with M 5.4 hit the area of Norcia (Perugia). In the consecutive month thousands of aftershocks took place along the NW-SE fault system extended for about 30 km, as shown in the following graphic compiled by the *Instituto Nazionale di Geologia e Vulcanologia* (INGV), figure 2.2.

On the 24th August, the number of fatalities reached 299, with 400 injured and more than 4000 left homeless. Although Amatrice is a small town with 2650 habitants, in August the number of population increased due to tourism and, allied to this fact, the earthquake occurred during the night, causing a high number of victims. In this town, the number of victims was the highest of all villages, reaching the number 200.

Nevertheless, INGV had already signalized this area with high seismic risk because the 24th earthquake occurred in a "gap" between two earlier damaging events, the 1997 M 6.1 Umbria-Marche earthquake and the 2009 M 6.1 L'Aquila earthquake, figure 2.4.

The aftershock pattern is consistent with the NW-SE trending fault, as expected, and the following figure 2.3 shows the pattern five days after the seismic event.

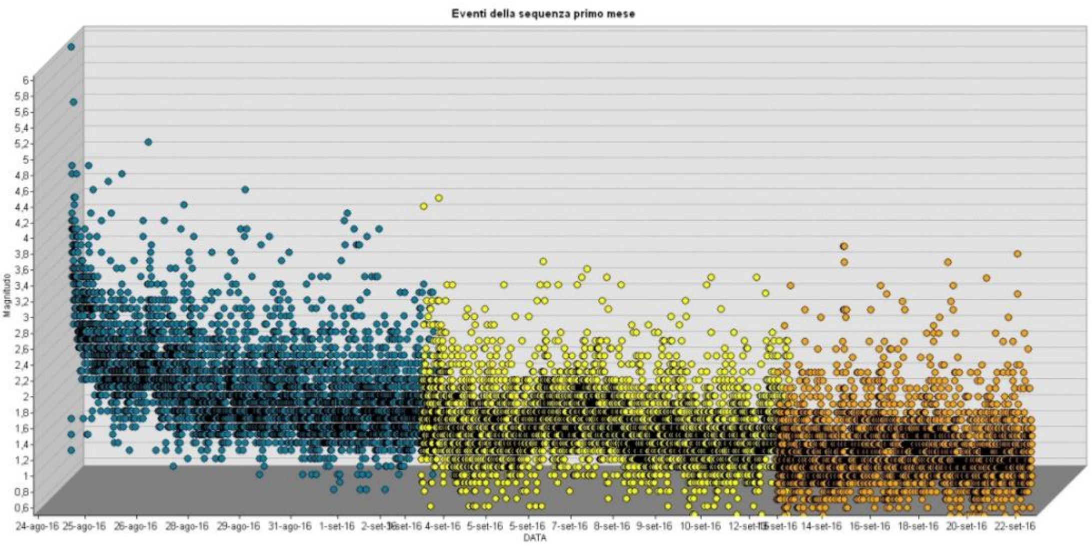


Figure 2.2-Earthquake's chronological sequence from 24th August until 23th September (source INGV)

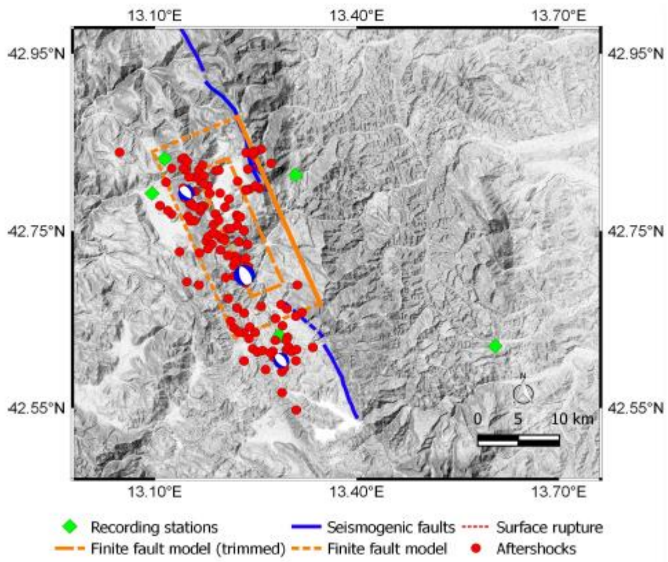


Figure 2.3-Map exhibiting the trending faults and the pattern created by the aftershocks (lined up with the NW-SE direction)

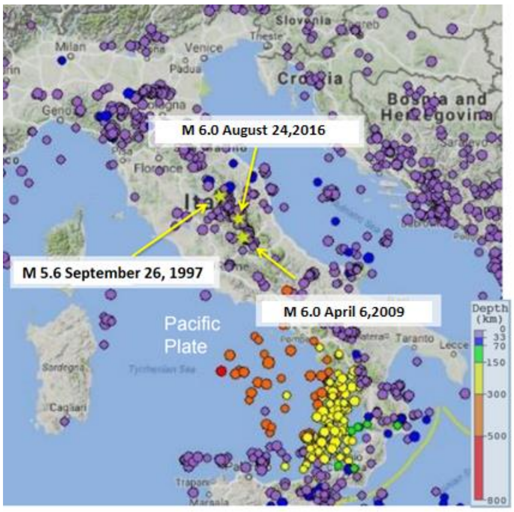


Figure 2.4-Locations of the epicentres of the 24th August and the previous events of Umbria-Marche 1997 and L'Aquila 2009.

The 24th August was 50 km south-southwest of the M 5.6 September 26, 1997 earthquake which killed 11 people, injured over 100 and damaged or destroyed more than 80000 homeless. In relation to the M 6.3 April 6, 2009 earthquake, it was 45 km north-northwest of the near L’Aquila and this earthquake killed 295 people, injured over 1000 and left over 55000 homeless.

2.1.1. The 26th and 30th October

In the following month, two more damaging events occurred on the 26th and 30th October. Regarding the first, the epicentre was located 20 km north from Norcia, with M 5.9 and the second in 30th October, 6 km north of Norcia, with M 6.5. The latter was the largest earthquake recorded in Italy since the M 6.9 Irpinia event in 1980, which killed 3000 people and left 300000 homeless. The three events have been caused by normal faulting, the prevalent style of faulting in the area, all of them having NW-SE strike and dip towards SW.
Although the 30th earthquake was more powerful than the 24th in terms of magnitude, this latest event claimed only one life, despite the significant level of damage observed, particularly in the Norcia region. The reason behind is the previous evacuation of the population after the 24th August and retrofit measures took by the town of Norcia after the 1979 Norcia and 1997 Umbria-Marche earthquakes. However, these two last seismic events will not the object of discussion in this work.

Table 1- Resume of the three earthquake parameters and two aftershocks (source INGV)

Date	Hour (UTC)	Latitude (N)	Longitude (E)	Depth (km)	M
08/24/2016	01:36:32	42.7	13.23	8	6.1
08/24/2016	02:33:28	42.79	13.15	8	5.3
08/24/2016	04:28:25	42.6	13.29	9	4.8
10/26/2016	19:18:05	42.92	13.13	8	5.9
10/30/2016	06:40:17	42.84	13.11	5	6.5

2.2. DAMAGE "GRADING" MAPS

With the purpose of assessing the damage after the earthquake, a damage map was elaborated by *Copernicus Emergency Management Service Mapping*, for all the affected areas. Here, will be presented damage maps for Amatrice, Pescara del Tronto and Arquata del Tronto. These maps, along with 58 others from different areas, supported the authorities in making informed decisions about the locations in the most urgent need of intervention. Copernicus system was activated by the Italian Civil Protection Department (DPC) a few hours after the occurrence of the earthquake. At this point, the authorities had to understand the gravity of the damage in the various affected areas, the more urgent cases to send direct rescue teams and locate the blocked roads and alternative accessing paths. In support of this kind of decisions the Copernicus EMS provides damage "grading" maps made by comparing pre- and post-earthquake satellite images.

2.2.1. AMATRICE DAMAGE MAP

From these maps, it is obvious the total destruction of Amatrice centre, with all of the buildings destroyed or highly destroyed. In the suburbs, the damage level is inferior although some reinforced concrete and masonry buildings totally collapsed.

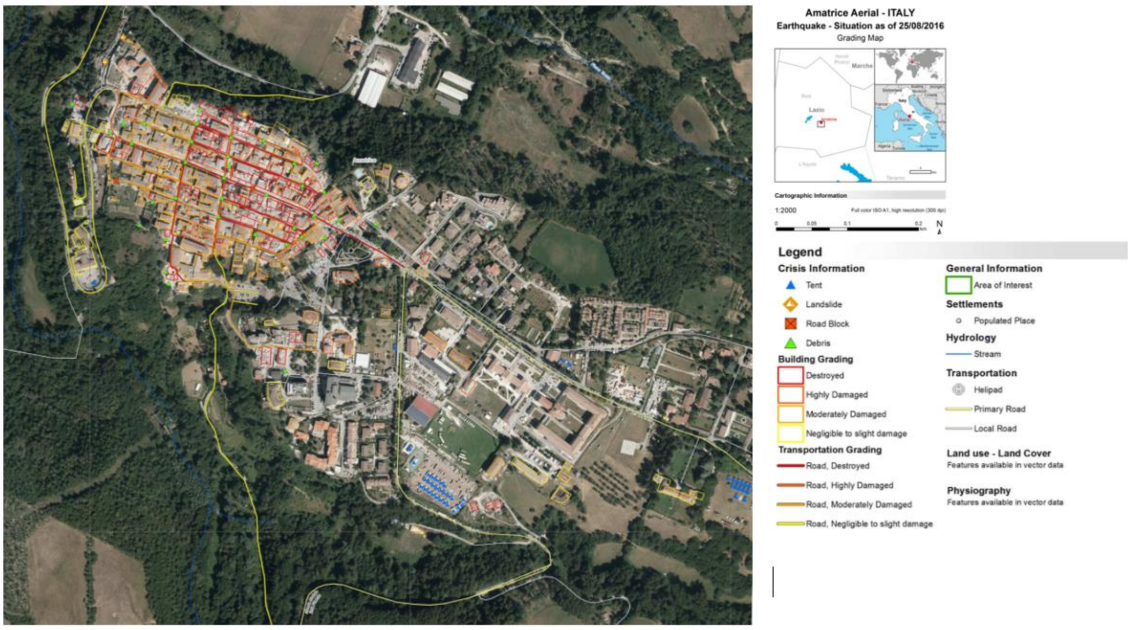


Figure 2.5-Copernicus damage map for Amatrice centre

2.2.2. Pescara del Tronto damage map

In Pescara del Tronto, a village much smaller, exhibits levels of destruction similar to Amatrice, the majority of the structures collapsed, leaving the village completely ruined.

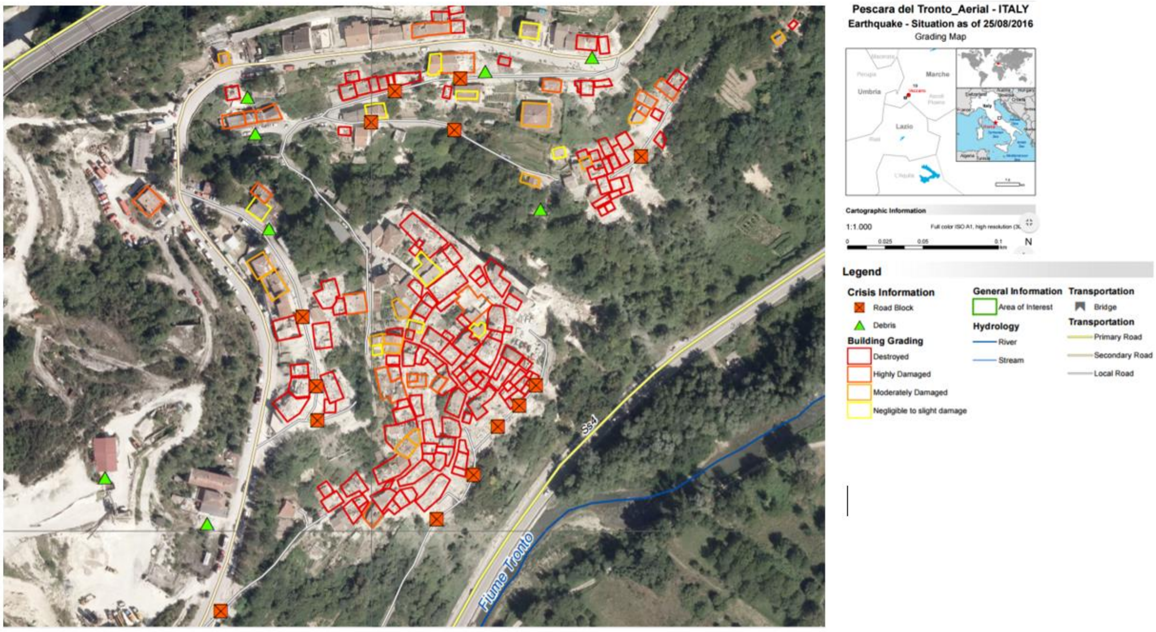


Figure 2.6-Copernicus damage map for Pescara del Tronto

2.2.3. Arquata del Tronto damage map

In Arquata del Tronto the level of destruction was slightly minor compared with the Pescara del Tronto and Amatrice. Still full and partial collapses of masonry buildings on top of the ridge were reported.

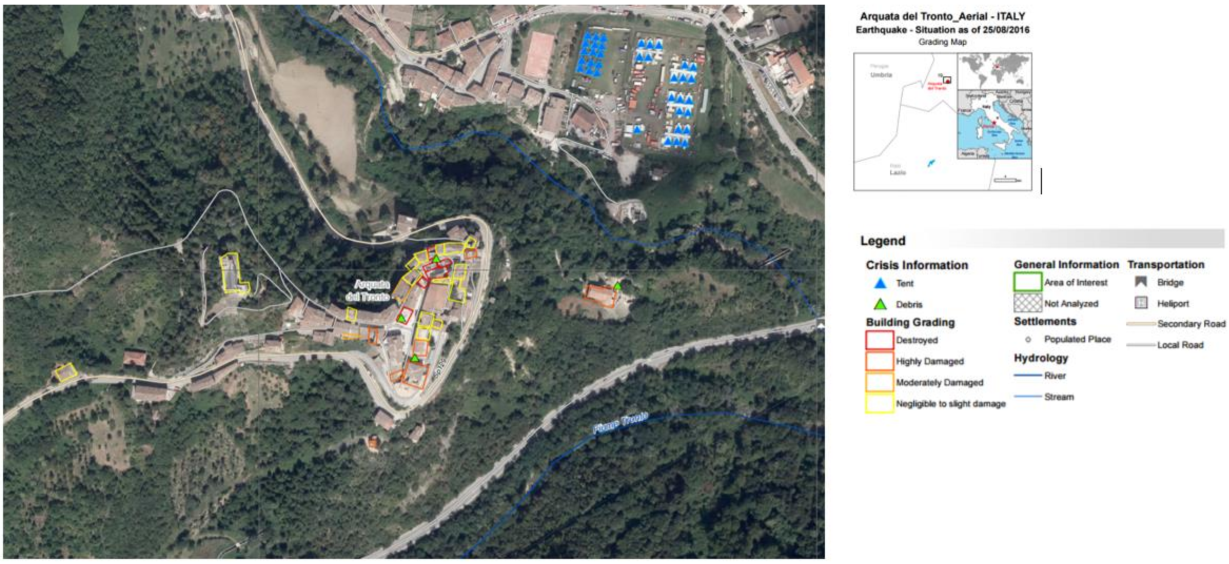


Figure 2.7-Copernicus damage map for Arquata del Tronto

2.3. DAMAGE TO BUILDINGS

The buildings that were well designed according to the seismic codes and well-built, generally performed satisfactorily and saved many lives. The dominant building types in the affected area are unreinforced masonry with two or three floors, some of which date back to the 13th century, and reinforced concrete buildings, erected since the early fifties with two to five floors. According to the *Istituto Nazionale si Statistica* (ISTAT) census 2011, 5% of buildings are reinforced concrete structures, 86% masonry structures and 9% structures of unspecified type.

About 70.8% of the 20000 residential buildings of the municipalities affected by the earthquake were built before 1971, when some anti-seismic regulations came into force. Before the earthquake, in 2011, 80% of the built was classified as excellent or in good conditions, and only 1.5% classified as degraded. The following figure 2.8 compiled by ISTAT shows the map of the seventeen affected municipalities and the locations of its public buildings, such as, hospitals, schools, post offices, police stations and the dark green represents the small inhabited hamlets. Some of these public structures were severely damaged during the earthquake, especially in Amatrice, section 4.1.1.1.

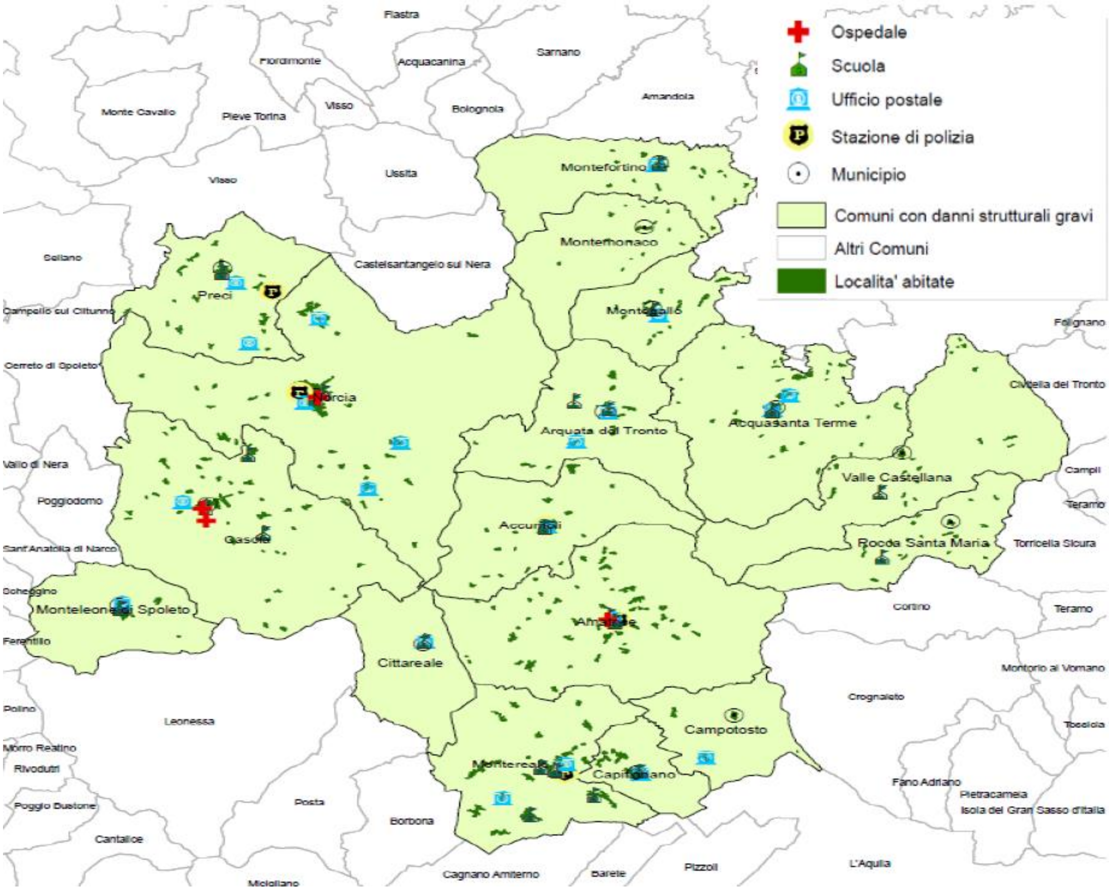


Figure 2.8-Damaged municipalities and locations of public buildings and small hamlets (source Istat)

2.3.1. MASONRY BUILDINGS

In the zone affected by the earthquake, the main construction technique used for unreinforced masonry buildings was a double leaf load-bearing wall, built with irregular stones (generally called *a sacco*) dressed in poor quality mortar. Usually there was lack of adequate connections between the two leaves and the space between leaves was filled with smaller rubble masonry. These buildings are characterized by flexible timber diaphragms and poor connections between walls and between walls and roof. The seismic vulnerability of this type of buildings is therefore very high because of their poor resistance to lateral loads.

Damage vary from a state of extensive cracking to global collapse depending on the quality of the masonry and structural irregularities. Most of failures consist in an out-of-plane overturning of the wall due to ineffective connection between diaphragms and orthogonal walls. Buildings with a better degree of connection of the walls exhibited in-plane behaviour. Observed damage due to in-plane behaviour are mainly diagonal and vertical cracks of both piers and spandrels panels.

Several ancient buildings in which the original timber floors and roofs were replaced with reinforced concrete elements collapsed partially or totally. The addition of reinforced concrete diaphragms did not seem to have been supported by effective connections or by improvements of the quality of existing masonry walls, for example with mortar injections. Without these measures, the insertion of concrete elements had, in fact, only increased the mass of the buildings making them even more vulnerable to earthquakes.

Another source of serious damage was the site effects because many buildings were situated in very steep lands causing the slip of the entire structure and, in several cases, the collision with buildings in lower soils.

Also, the modifications made along the years, like elevations and insertion of terraces at slab level, diminished the structural capacity of the existent buildings. These adjustments can be visually identified because of the presence of materials different from the original ones.

Regarding the town of Norcia, which had an epicentral distance of about 20 km, a very good seismic behaviour was exhibited only with minor damage reported, due to the severe retrofit measures implemented decades before. The out-of-plane mechanism was minimized by inserting steel ties, properly connected ring beams and stiffening the diaphragms. These construction details allowed the masonry walls to exhibit an in-plane behaviour and to increase their resistance.

2.3.2. REINFORCED CONCRETE BUILDINGS

The reinforced concrete buildings in the affected area are mainly multi-storey, usually four stories, framed structures with masonry infills. Some of these behaved well only exhibiting in plane failure of the infill panels and damage in the beam-column joint because they were constructed according to the seismic code. On the opposite scenario, many of the concrete structures were designed when seismic details were not legally prescribed and for this reason, the influence of the masonry infills on the global behaviour was neglected. As many of the construction dates from before 1970, many buildings were designed to bear vertical loads only.

Severe damage occurred to non-structural elements, in particular infills and internal partitions: external infills suffered widespread cracking in their plane, due to the lack of properly designed gap between them and the load bearing frame. Often infills were ejected due to out-of-plane failure because there was not an effective connection between them and the reinforced concrete frame. In other cases, occurred brittle failures caused by soft storey mechanism. Damage to structural elements are mainly brittle failures of columns and crisis of the beam-column joints due to a low shear resistance - too spaced stirrups or lack of them in the nodal zone.

Normally, reinforced concrete buildings performed better then masonry structures, also due to the ground motion characteristics, section 2.6.

2.3.3. Historical and Cultural buildings

The loss pattern once again highlights the fact that central Italy is characterised by the unfavourable combination of high seismic risk and large number of historic buildings. Regarding the historical/cultural heritage, according to *ISTAT*, the balance of damage drawn up by the Ministry of Cultural Heritage and Tourism and the *Protezione Civile*, counts 293 assets of cultural interest destroyed or severely damaged.

In Amatrice, half the façade of the 15th century church of Sant'Agostino has collapsed, figure 2.10, as well as other churches like Santa Maria del Suffragio, figure 2.14, and one museum. In the near villages like Accumoli similarly other churches suffered severe damage like the case of San Giovanni, where one load bearing wall partially collapsed, figure 2.11, and in the centre, part of the façade of the church disappeared figure 2.12.

By contrast, the 16th century bell tower remains standing and its clock froze at just 03:36, the moment the earthquake hit the town figure 2.9. In Norcia, the 14th century San Benedetto basilica partially collapsed during the 30th October, figure 2.13.

This pattern of damaged historic patrimony dates back many decades. For example and very briefly, in 1997 a M 6.1 earthquake that struck the Colfiorito basin (30 km north of Norcia) caused widespread and severe damage. The arched ceiling in the Upper Basilica of Saint Francis in Assisi was one of the structures unable to resist the shaking at that time. Another recent example is L'aquila which reconstruction and restoration of the historic city centre is still ongoing since 2009.

Figure 2.9-Bell tower showing the exact time of the catastrophe, 03:36

Figure 2.10-Sant'Agostino churche with its tympanum collapsed

Figure 2.12-Collapse of a masonry church, Accumoli

Figure 2.11-Collapse of San Giovanni church, Accumoli

Figure 2.14 -Church Santa Maria del Suffragio, Amatrice red zone

Figure 2.13-Church San Benedetto, Norcia after the October 30, 2016

2.4. LANDSLIDES AND ROCK FAILURES

The reconnaissance map of landslides and rock failures due to the seismic event was conducted by three teams: ISPRA (Italian Institute for Environmental protection), CERI (Geological Risks of Sapienza University) and GEER (Geotechnical Extreme Events Reconnaissance Association), figure.2.15, and, in the overall, more than 140 instabilities were reported. Indeed, land sliding and rock falling represents an important collateral seismic hazard in the Apennines chain, where the widespread presence of weak geological materials results in a high susceptibility to failure even under non-seismic conditions.

2.4.1. ROCK FAILURES

Rock failures were associated to wedge slides, toppling and slides in intensely fractured or degraded rocks. The typology of rocks in question involved flysch units, sandstone layers interleaved with marls layers, and carbonatic units, mainly limestone. Most of the rock fall happened when isolated units of limestone dissociated from the mother rock and ended up in the highway, figure 2.16. It is worth to mention, that the majority of the slope protection such as rock bolts, rock fall nets and tiebacks had a positive action in preventing more damage. With the closure of many accesses the rescue of the population was not possible, although some of these were already closed before the event.

2.4.2. LANDSLIDES

Regarding the landslides, most of them seemed to have coincided with retaining wall failures. With graben displacements which accompany the retaining wall failures, at first sight, is difficult to know if the soil deformation provoked the collapse of the retaining wall (global failure below the wall) or the opposite, the retaining wall provoked the soil deformation (local failure behind the wall). To solve this question, engineers should assess the mode of retaining wall failure and depth and geometry of displacements in the graben. “Most of the soil displacements appear to be caused by failure of the retaining wall due to the limited extent of the soil displacement, suggesting that the deformations correspond to a local slope failure behind the wall” (source GEER).

Pescara del Tronto was one of the villages that suffered more landslides and retaining wall failures, including the largest landslide that happened on the east zone figures 2.17-2.18. The landslide itself was shallow, being the width of sliding soil less than one meter.

Regarding the damage observed in Accumoli, the instabilities were mainly significant deformation and cracking in the soil, in the eastern area and no landslides were reported. Figure 2.20 exhibits a cracking pattern that, at first sight, seemed due to a landslide but later was found to be the result of a retaining wall failure. The concrete wall rotated 3.5 degrees, with a horizontal offset of 57 cm and vertical offset of 18cm. The soil graben measured 2.7 meters wide and the soil settled approximately 50cm.

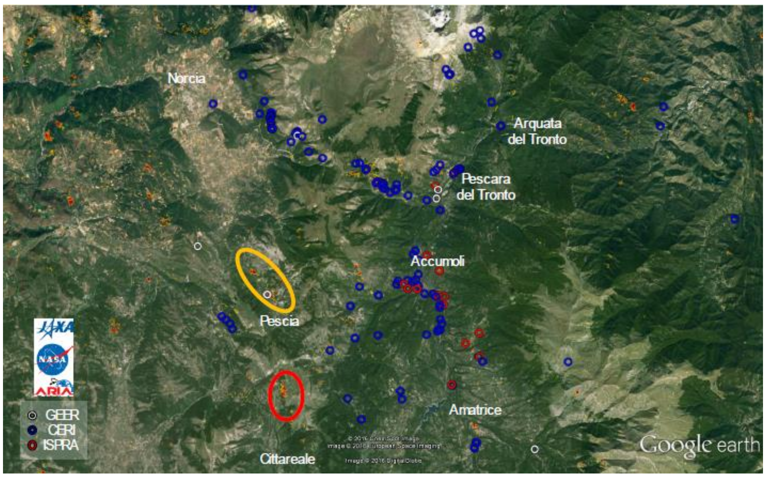


Figure 2.15-Mapped rockfalls and landslides from ISPRA (red circles), CERI (blue circles), and GEER (white circles)

Figure 2.16 -Rock fall on SP477. The block is 2 meters unit of limestone that desagregated from the bedrock

Figure 2.17 -Screenshot of the 3D model of Pescara del Tronto (source GEER)

Figure 2.18-3D model image of largest landslide in Pescara del Tronto, in the east part (source GEER). It is about 30 meters high and 75 meters wide

Figure 2.19-3D model image of two slope failures near the largest one, Pescara del Tronto (source GEER) .

Figure 2.20-Retaining wall failure and rotation of the concrete wall causing significant cracking in the soil immediately behind (graben), (source GEER)

2.5. Seismicity of the Region and the 24th August Earthquake

Central Italy is a high seismic hazard region caused by the collision between the African and Eurasian tectonic plates. The African plate is subducting bellow the Eurasian plate producing continuous increase of stresses over a series of active faults in the central Apennine chain. Current motion of the African plate with respect to the Eurasian plate is north-northwest at about 7 mm/yr. Evidences of this activity have been proven by its seismicity: strong earthquakes, as well as minor sequences and seismic swarms of low energy along NW-SE trending ruptures. Seismicity can be defined as “probability in a given area and in a certain interval of time of an earthquake exceeding a certain threshold of intensity, magnitude or peak ground acceleration (PGA)”, (source *Protezione Civile*). Also, geological data points out normal faults activity during the Quaternary (Galadini and Galli, 2000; Boncio et al., 2004a; Roberts and Michetti, 2004).

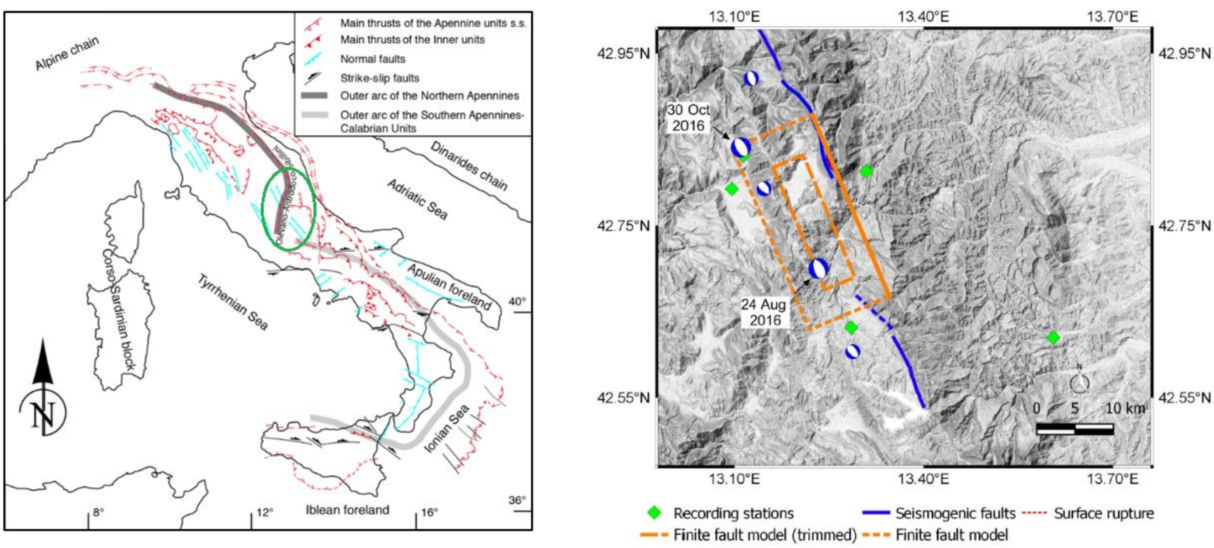


Figure 2.21-From left to right: Italy's map of existent faults and thrusts. The green circle indicates the faults responsible for the 24th August. On the right, a close up to these faults, Laga Mts. and Mt. Vettore, with the epicentre of the 24th August.

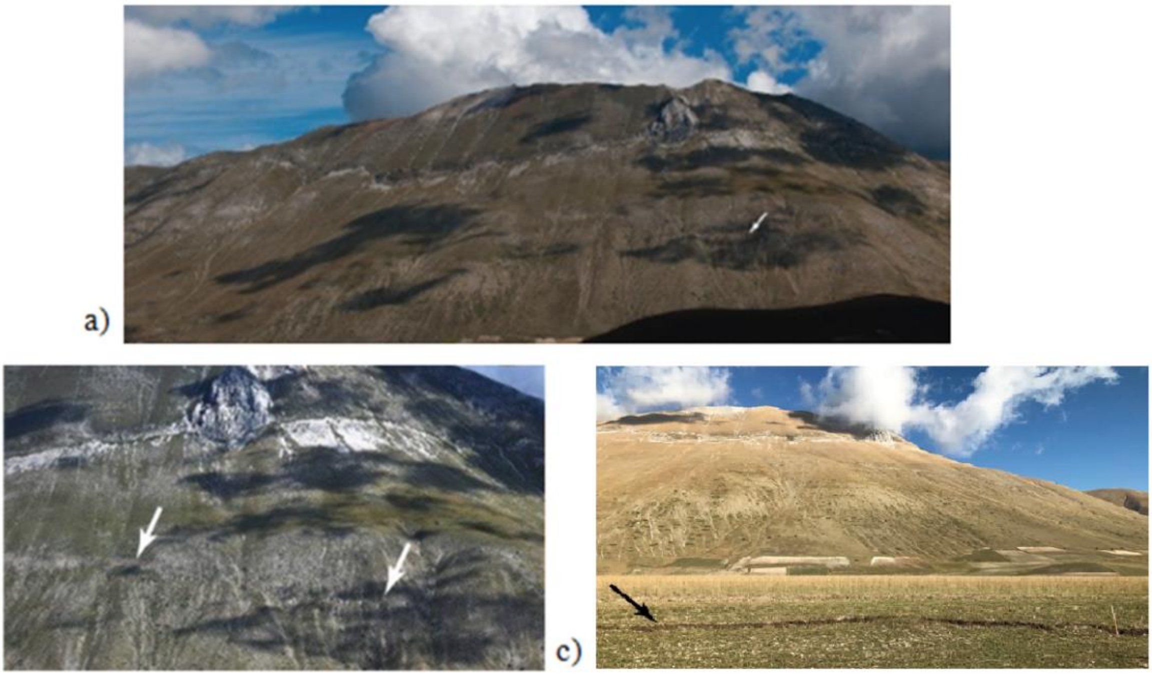
“The earthquake occurred on a NW-SE (strike 156 g) trending normal rupture with dip SW (50g), manifestation of the extensional tectonic regime ongoing in the central Apennine chain” (source INGV). The epicentre is located between two silent faults, the Laga Mts. and Mt. Vettore, and the strike of the fault is lined up with Mt. Vettore in the north direction and with the Laga Mts. in the south direction. Therefore, it seems that the 24th August earthquake ruptured both faults on a multi-segment surface rupture. These normal faults are visible in the abovementioned map of Italy’s map of existent faults and thrusts, in the green circle.

The surface rupture took place over 4.8 km along Mt. Vettore fault, being the maximum measured displacements on the primary rupture of 35 cm with an average of 12 cm. These features are coincident with a 6.1 magnitude earthquake (Wells and Coppersmith, 1994 Empirical relationships among magnitude, rupture length, rupture width, rupture área and surface displacement).

The following sub-chapter characterizes these two faults although, in this area there are more four faults believed to have produced the earthquake including the Upper Aterno Valley and Paganica faults, responsible for the destruction in L’aquila in 2009.

2.5.1 Mt. Vettore Fault

The Mt. Vettore fault strikes NNW-SSE to NW-SE and dips WSW to SW with a surface length of 27 km and is considered by many authors as active. Evidences of this activity are the "displacements of the alluvial fan in the northern sector of the Castellucio basin due to motion of minor fault sections" (Galadini and Galli, 2003). The alluvial fan dates between 23000 and 3200 BP (years before the present) and the associated faults from the Pleistocene-Holocene until 3200 BP with a slip of 0.36 to 0.62 mm per year. This fault has been "silent" for more than 2500 years (Galadini and Galli, 2003) because no historical seismic events are known to be related to this fault, until now.



a) b) c)

Figure 2.22 -Mt. Vettore fault. a) and b) the biggest scarp of the fault called "Cordone del Vettore", belonging to the SW slope and indicated by the white arrows.c) 10-20cm displacemente caused by the seismic event on 30th October 2016

2.5.2 Laga Mts. Fault

The Laga Mts. Fault strikes NW-SE and dips SW along a surface rupture of 26 km. This fault activity reassembles to the Late Pleistocene and Holocene in the southern segment (Campotosto plateau and Vomano valley) and some authors admit the possibility of the activity migration from the north to the south. It is believed that the northern sector is capable of producing 5.9 magnitude earthquakes along an 8 km rupture surface, while the southern sector suggests 6.5 magnitude events along a 18 km surface. For this reason Akinci et al. (2009) consider that there are two different fault sectors and the background seismicity is coherent with his theory: the 2009 L'aquila event was associated with the southern sector while the last year event was related with the northern sector. Regarding the seismicity history only one destructive event can be associated with this fault, the earthquake of 1639 in Amatrice with 6.2 of magnitude.

Figure 2.23-Laga fault, southern sector called Campotosto plateau. The arrows show the detected bedrock fault scarps

2.5.3 Hazard Seismic Map and Macroseismic Intensity

Italy has a high seismic hazard not only due to the frequency and intensity of earthquakes, but also due to the high vulnerability of buildings, high exposure due to population density and historical, artistic and monumental heritage. The victims, damage to buildings and direct and indirect costs expected after an earthquake are very high. In this area, PGA values expected, with a probability of exceedance of 10% in 50 years, are higher than 0.25 g.

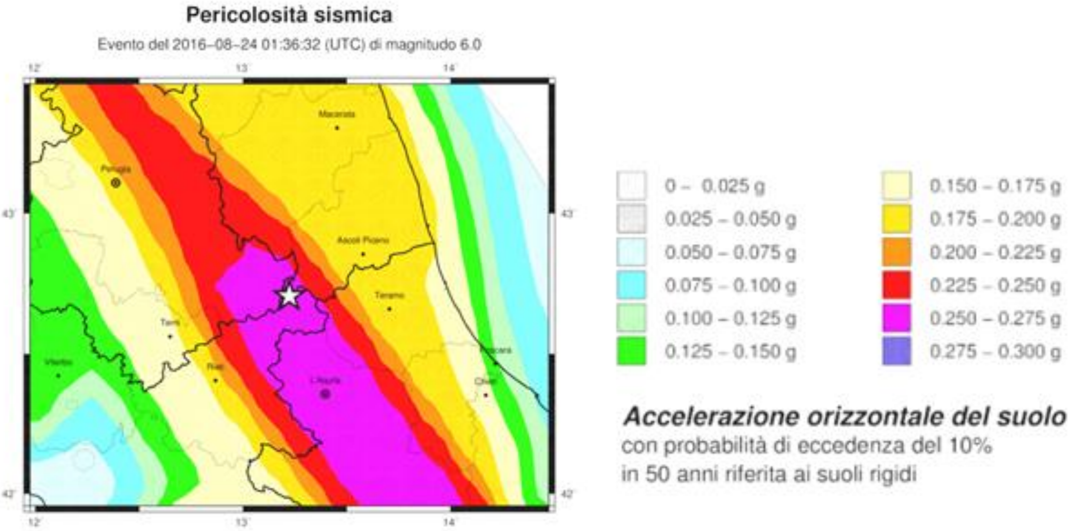


Figure 2.24-Hazard seismic map of Central Italy and the epicentre of the 24th August

Seismic intensities were very high, IX MCS, in Pescara del Trontodo t and Accumoli, being the maximum recorded PGA 0.43g. These high values and the very intense effects can be explained by soil amplification (increase of ground motion at the soil surface compared to motion at bed rock) and epicentre vicinity to the villages and to the surface. Soil amplification explains why many times damage can be more severe in some buildings in comparison to their neighbours. Therefore, a more detailed analysis on this matter will be elaborated in chapter 4.

However, is difficult to estimate with accuracy the real level of intensity in some cases due to the presence of different typologies of construction, high state of degradation before the occurrence of the earthquake and, especially in Amatrice, the sharp different between historic centre damage and periphery damage.

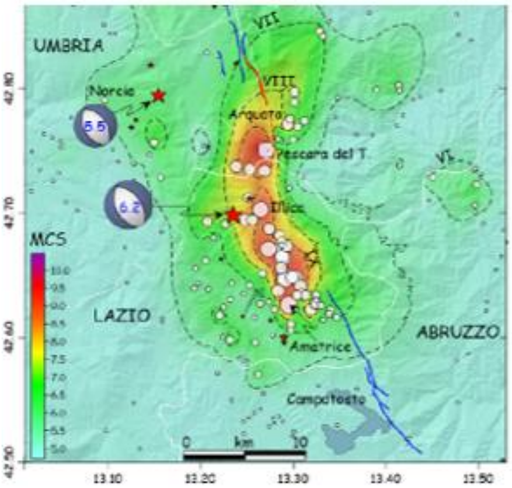


Figure 2.25-Macroseismic intensity in the most damaged cities according to Mercalli's scale (Galli et al.,2016). The stars represent the epicenter of the main and aftershocks and the blue lines the silence faults, Vettore and Laga. White circles are proportional to the site intensity. The isoseismical lines from VI to IX MCS are represented with a black hatching. On the right, a street in Amatrice parallel to Corso Umberto I.

2.6 ITALIAN SEISMIC CODE AND RECORDED SPECTRUM

With the purpose of obtaining a more comprehensive evaluation of the 24th August earthquake with the current seismic design provisions for Amatrice, a comparison was made between the acceleration response spectrum of the recorded earthquake and the Italian design code, NTC 2008, figure 2.26. The acceleration response spectrum associated to the horizontal ground motions during the event was recorded by four stations with lowest epicentral distance (AMT, NRC, RM33 and SPD) and compared with the elastic spectrum (5% damping) with four different return periods (TR): 50, 475, 975 and 2475 years and for type B soils. The distances of those stations are respectively: 8.900 km, 15.200 km, 21.700 km and 23.600 km.

Maximum recorded accelerations are 0.43g in Amatrice station and 0.37g in Norcia station, although their level of damage is quite distinct, as aforementioned. The more distant the stations are from the epicentre, the lower PGA values are recorded, which is consistent with the known fast-attenuation features in Italy (*leggi di attenuazione).*

From the observation of the spectrums it can be concluded that the higher accelerations are in the range of 0.2-0.3s, in relation to other periods. This range corresponds, grossly, to rigid structures (typically low structures) consistent with most of the built in Amatrice and small villages, which could explain their severe damage. To corroborate this theory, there is the example of one high and flexible reinforced concrete building in Amatrice centre that remained standing, while the surrounding structures are destroyed, because of its higher period, 0.4-1.0s, corresponding to lower accelerations values. Unfortunately, this building did not resist the second main earthquake in 26th October due to its significant level of damage.

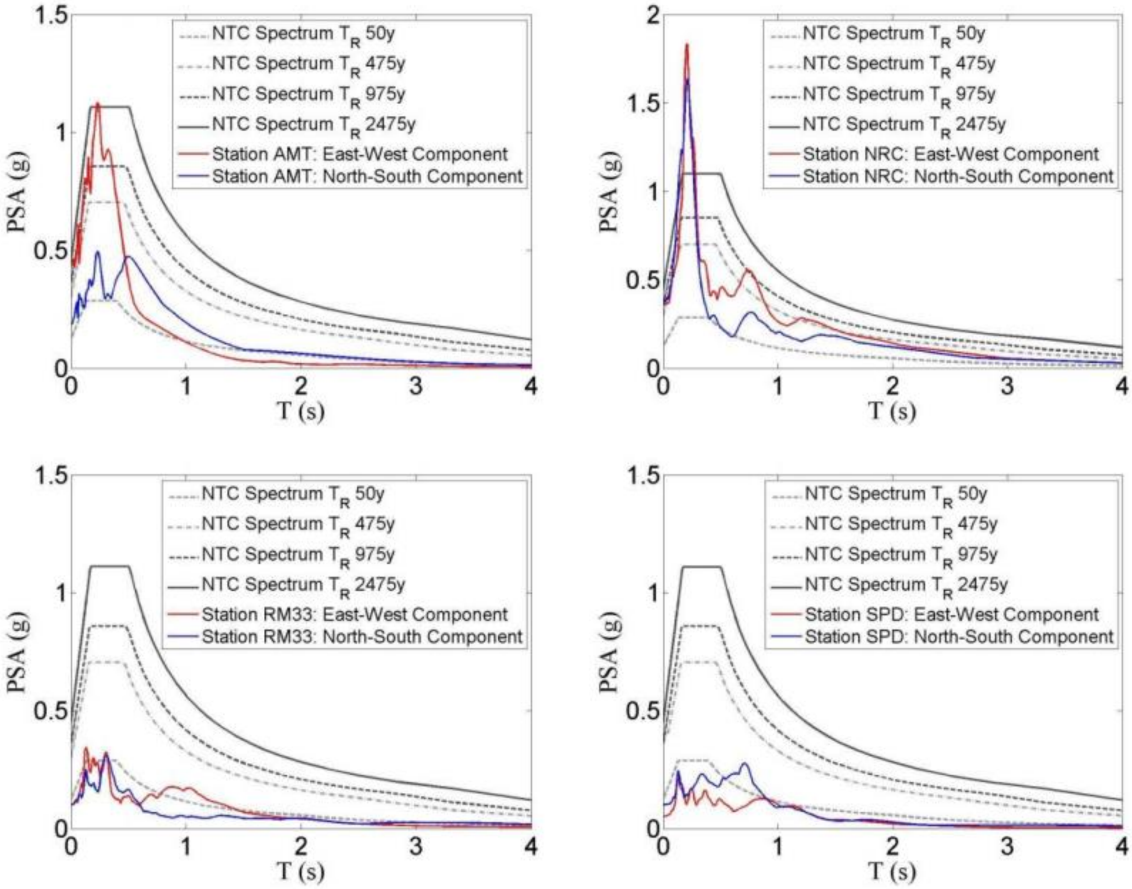


Figure 2.26 -Comparison of the four closest-to-rupture stations' horizontal components PSA response spectra with the Italian code elastic response spectrum at various return periods

In conclusion, the earthquake excited mainly short periods, coincident with the periods of masonry buildings, and, as reinforced concrete buildings have higher period range, they were not so affected.
Current buildings are dimensioned for a 475-year return period (Ultimate Limit State), only special buildings can be dimensioned for 975 years of return period. From these acceleration spectrums, in the nearest stations (Amatrice and Norcia), the horizontal components exceed the limit of 2475 years (Limit State Design), meaning that old unreinforced masonry buildings could never resist this event. Therefore, it is important to design new buildings with appropriate spectral acceleration levels compatible with possible future scenarios.

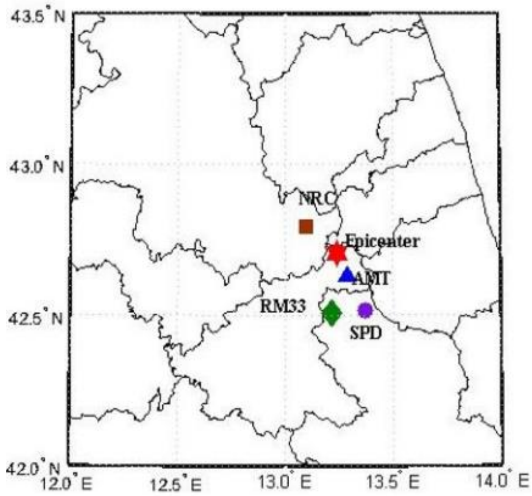


Figure 2.27 -From left to right: map of the four stations with lowest epicentral distance. On the right, standing reinforced concrete building in Amatrice centre

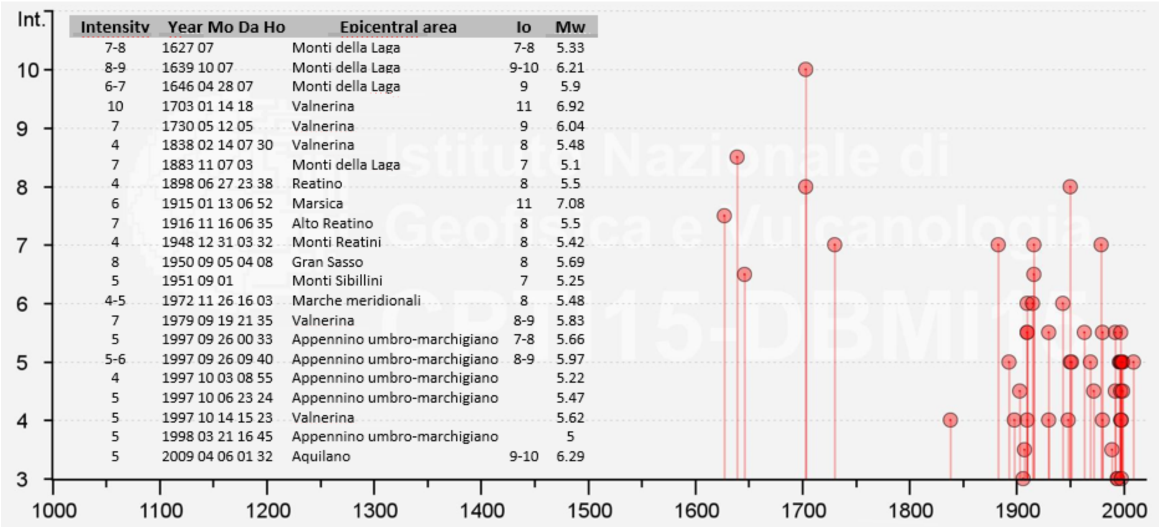
2.7 Seismic History of the Region

“In 2500 years, Italy has been hit by over 30000 medium to strong earthquakes measuring more than grade IV-V on the MCS, and by around 560 events of an intensity equal to or higher than grade VIII MCS. In the twentieth century alone, seven earthquakes had a magnitude of 6.5 or more (grade X and XI MCS)”, source *Protezione Civile*.

Regarding in detail the Amatrice sector, the following earthquakes were reported and are well documented in the literature: 1627 (M 5.3), 1639 (M 6.2), 1672 (M 5.3) and 1703 (M 6.9) figure 2.28. The latter was particularly devastating destroying large part of the villages in the surroundings of Norcia. More recently the earthquake in 1915 in Avezzano, province of Abruzzo, killed 30000 people, 90% of the population, in 1976 in Friuli Venezia Giulia 1000 were killed by the natural disaster and, in 2009 in L’aquila 300 fatalities were the result of an M 6.3 earthquake.

To analyse more profoundly this background seismicity after the August event, a study was conducted by the INGV during the last year, figure 2.29. In this study was concluded, very briefly, that more than 8900 earthquakes struck the central area of the Apennines chain since 1981. Figure 2.29 right shows the ones with M>5. The data came from two different catalogues: the Catalogue of the Italian Seismicity (CSI, Castello et al., 2006) for the period 1981–2002, and the *Bollettino Sismico Italiano* of INGV for the period 2003–2016. Figure 2.29 left represents earthquakes with magnitude superior to 3.5 and it can be concluded by this graphic that this sector has been constantly hit by medium-high occurrences during the last 35 years.

Figure 2.30 reports the epicentre locations and magnitudes of the most powerful earthquakes of the last 50 years in Italy.



Intensity	Year Mo Da Ho	Epicentral area	Io	Mw
7-8	1627 07	Monti della Laga	7-8	5.33
8-9	1639 10 07	Monti della Laga	9-10	6.21
6-7	1646 04 28 07	Monti della Laga	9	5.9
10	1703 01 14 18	Valnerina	11	6.92
7	1730 05 12 05	Valnerina	9	6.04
4	1838 02 14 07 30	Valnerina	8	5.48
7	1883 11 07 03	Monti della Laga	7	5.1
4	1898 06 27 23 38	Reatino	8	5.5
6	1915 01 13 06 52	Marsica	11	7.08
7	1916 11 16 06 35	Alto Reatino	8	5.5
4	1948 12 31 03 32	Monti Reatini	8	5.42
8	1950 09 05 04 08	Gran Sasso	8	5.69
5	1951 09 01	Monti Sibillini	7	5.25
4-5	1972 11 26 16 03	Marche meridionali	8	5.48
7	1979 09 19 21 35	Valnerina	8-9	5.83
5	1997 09 26 00 33	Appennino umbro-marchigiano	7-8	5.66
5-6	1997 09 26 09 40	Appennino umbro-marchigiano	8-9	5.97
4	1997 10 03 08 55	Appennino umbro-marchigiano		5.22
5	1997 10 06 23 24	Appennino umbro-marchigiano		5.47
5	1997 10 14 15 23	Valnerina		5.62
5	1998 03 21 16 45	Appennino umbro-marchigiano		5
5	2009 04 06 01 32	Aquilano	9-10	6.29

Figure 2.28-Graphic with important earthquakes since 1627 until 2009 with the following informations: year, month, day and hour, intensity, epicentral distance and magnitude

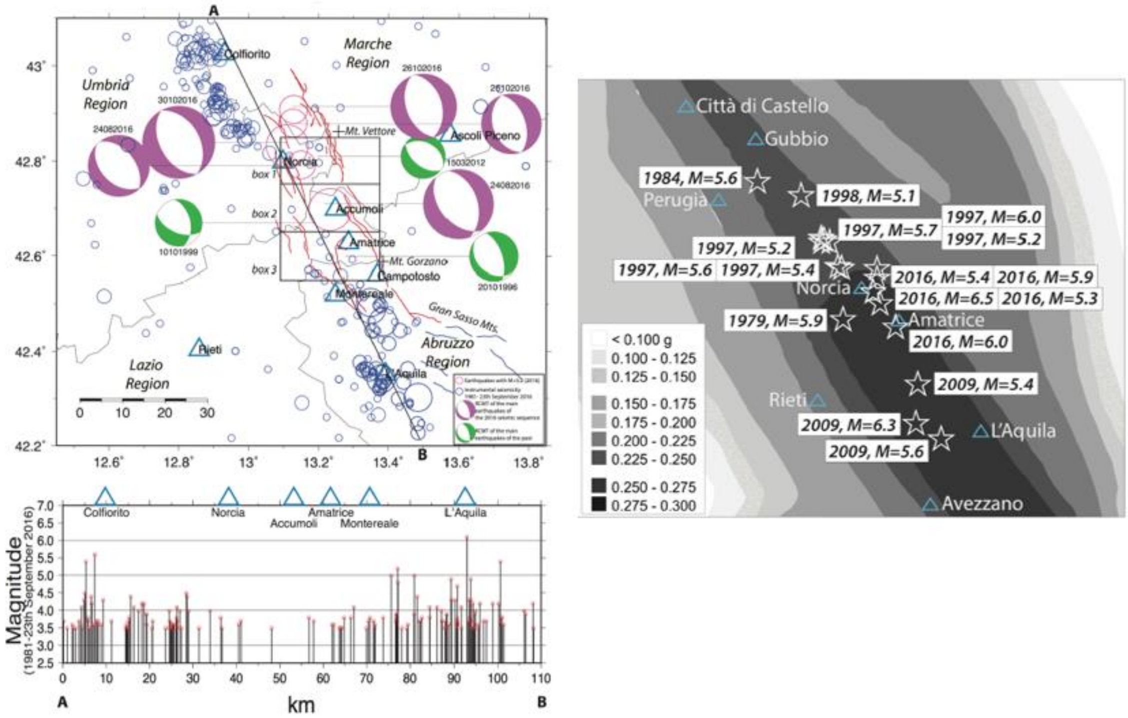
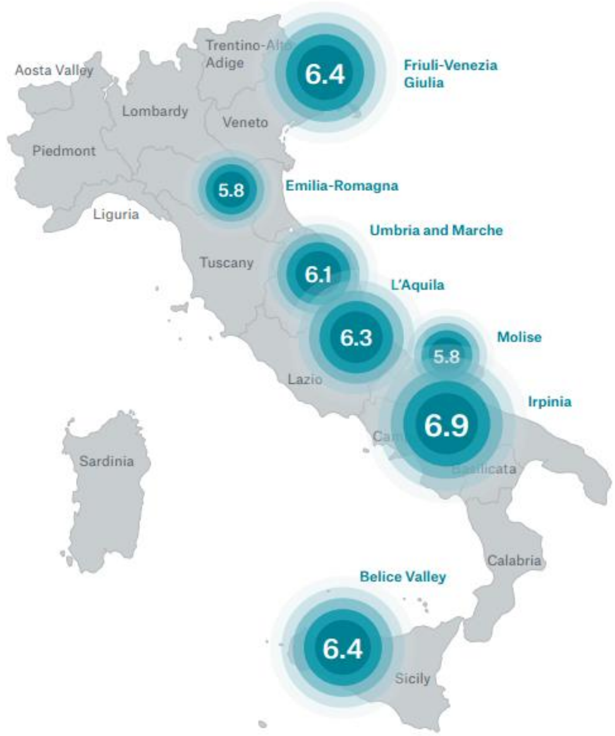


Figure 2.29-From left to right : Patterns of seismicity M >3.5 in the central Apennines, Italy, from 1981 to the 23th August 2016. Red circles represent earthquakes with M>5 and blue circles the remaining. Red lines represent the faults mapped by EMERGEO Working Group, 2016. On the right, epicenters of the earthquakes with M> 5.0 related to the most important seismic sequences occurred in the target region over the last 35 years and the maximum expected values of peak ground acceleration.



Earthquake
(M_W = moment magnitude)

1968	M_w 6.4	Belice Valley
1976	M_w 6.4	Friuli-Venezia Giulia
1980	M_w 6.9	Irpinia
1997	M_w 6.1	Umbria and Marche
2002	M_w 5.8	Molise
2009	M_w 6.3	L'Aquila
2012	M_w 5.8	Emilia-Romagna

Figure 2.30-Italy's most powerful earthqaukes in the last 50 years

3
FAILURE MECHANISMS

3.1. MASONRY STRUCTURES

Failure mechanisms in masonry structures are divided in two groups, the first damage mode and the second damage mode. The first is produced by seismic actions perpendicular to the wall, out-of-plane damage, and the second one is caused by seismic actions in the plane of the masonry wall, usually producing crack patterns. Seldom has this type of mechanism provoked the collapse of the structure, being the out of plane mechanism the one responsible for most of masonry failures.

It is generally recognized that a satisfactory seismic behaviour is obtained when the out-of-plane collapse is prevented and in-plane strength and deformation capacity of walls can be fully explored, because masonry walls are less resistant to perpendicular actions. The stiffness of the wall is by far fewer in the perpendicular direction than in the plane direction. This behaviour is the so-called box type structural system, composed of vertical structural elements, walls, and horizontal structural elements, floors and roofs. In this system, vertical loads are transferred from the floors, acting as rigid horizontal diaphragms, to the bearing walls, and from the bearing walls, acting as vertical compression members, to the foundation system. Hence, box action results in limiting the deformations imposed to masonry during an earthquake, preventing extensive damage and collapse. However, old masonry structures seldom satisfy the conditions of the box behaviour: floors and roof are rarely well connected with the walls and do not behave as rigid diaphragms in their plane, the connections between walls are not effective, and many times openings located close to the corners of buildings decrease the structural capacity. Also, previous non-repaired damage, lack of maintenance, decay of materials and changing in time, obviously aggravate the effects of a seismic event. In conclusion, the in plane behaviour respects the box structural system unlike the out of plane behaviour. The next figure illustrates the favourable box behaviour and the expected crack patterns and deformation of walls due to seismic action. Needless to say, that it is not always easy to distinguish the occurrence of a specific type of failure mechanism, since many interactions between them may occur.

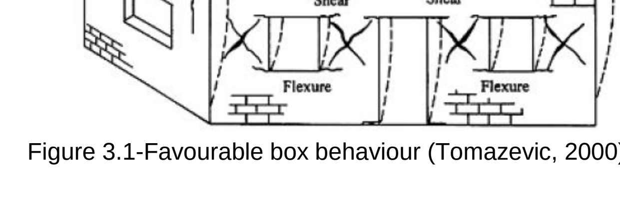


Figure 3.1-Favourable box behaviour (Tomazevic, 2000)

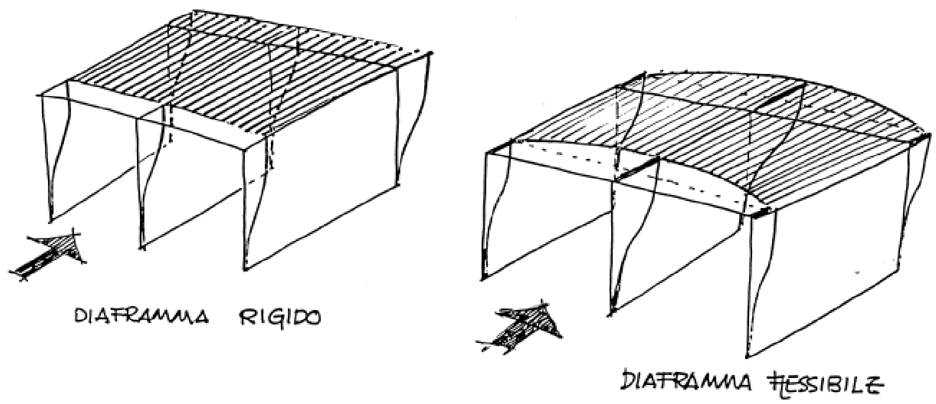


Figure 3.2-Illustration of the differents behaviour of the rigid and flexible diaphragms. In the case of the flexible diaphragm, it has insufficient stifness in its plan to distribute the horizontal inertial forces to the vertical systems

On average, masonry buildings perform very poorly in earthquakes. In Italy, masonry buildings are characterized by an intrinsic vulnerability to the seismic action, especially in historical centre and were considered during a long time as “minor” constructions, without any preservation principle applied, causing major damage under earthquake’s occurrence.

Masonry buildings often result from heterogeneous constructive phases due to extensions occurred in the past, as mentioned before. Also, the lack of maintenance is an important factor responsible for the degradation and increase of vulnerability of masonry buildings.

3.1.1. Masonry Typologies and Quality

The construction type, quality and state of preservation of masonry play a fundamental role in determining the capacity of a building to sustain seismic actions. The structural performance of a masonry can be understood once the following factors are provided (Binda, 2000): its geometry, the characteristics of its masonry texture (single or multiple leaf walls, connection between the leaves, joints empty or filled with mortar), physical, chemical and mechanical characteristics of the components (stones, mortar) and the characteristics of masonry as a composite material. When considering the mechanical and the physical behaviour it should be present that masonry is a heterogeneous material, and for this reason, there are many masonry typologies. The differences between them lay, not only on the use of materials according to the local’s possibilities, but also on the construction’s technique. This being said, the behaviour of masonry highly depends on the principles of the construction.

Masonry is characterized by its composite character (stone or brick in combination with mortar joints), a brittle response in tension (with almost null tensile strength), a frictional response in shear (once the limited bond between units and mortar is lost) and anisotropy (response is highly sensitive to the orientation of loads). Good quality masonry should follow the next principles, according to the Italian Seismic Code (OPCM, 2003): “stones or brick laid in horizontal courses, vertical mortar joints not-aligned, use of almost square-shaped and big stones of large size, limited volume of mortar as compared to the volume of bricks or stones, in case of multi-leaf masonry, leaves transversely connected (headers) figure 3.4 left and sufficient mechanical properties of mortar and bricks or stones”.

One fundamental characteristic of a masonry wall is the monolithic behaviour in the lateral direction, under the action of horizontal forces or eccentric vertical forces. Usually if the wall is made by small pebbles or by two external leaves not well connected (rubble infill) the brittle collapse is expected, figure 3.4 right. The experience dictates that in several cases the ability of the masonry to open and reconnect immediately after without collapse is an important “defence”, to avoid damage. This capacity is prevented in some buildings due to the introduction of horizontal constraints (e.g. ring beams) resulting in the explosion of external leaves and/or cracking. The energy always dissipates trough the weakest element.

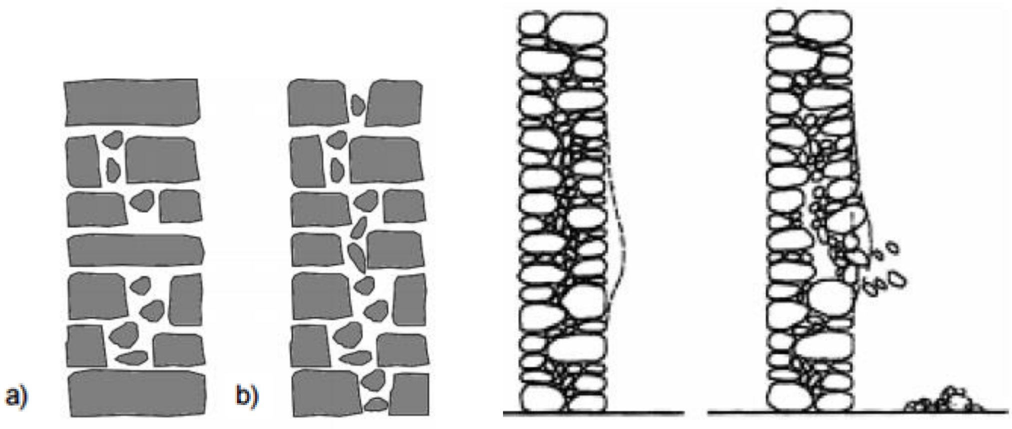


Figure 3.3-From left to right: a) example of a double leaf masonry connected by headers and b) example of a masonry fragile to earthquakes. On the right, the collapse of a double leaf masonry due to a non monolithic behaviour, sujecetd by horizontal or eccentric vertical forces

3.1.1.1. Possible Typologies of Masonry in Central Italy

A classification of masonry typologies was proposed by Speranza in (Speranza, 2003) after a data collection from the Umbria-Marche event in 1997, figure 3.5, and is usually applicable to a extensive quantity of mediterranean and european historic centres. These types of masonry were also found in the city of L’Aquila (D’Ayala, 2010) later the 6 April event, as well.

A1-Solid masonry constituted by long squared shaped stones disposed in horizontal courses. The connection along the thickness of the wall is declared good.

A2- Two wythes of squared dressed stones, with smaller constituents along the thickness of the wall. The infill (smaller constituents) between the two wythes is coherent and the attachment in the thickness is considered as medium.

B1-Mixed masonry limestone in long dressed elements and little squared shape stones and rubble. The final outcome is layers of long elements disposed along the bedding surface interleaved with elements along the thickness, headers. Rubble infill is used to fill in voids between stones. The global connection along the thickness is declared weak.

C1- Masonry constituted by rubble in large quantities, with layers practically horizontal. The overall layout is constituted by three or four leaves of little stones, interleaved with layers of larger elements. In the cross section is visible small stone elements and big quantity of weak mortar. The global connection is similar to B1, weak connection.

C2- Masonry constituted by small size rubble in large quantities, with layers less horizontal than C1. The little sizes of the elements and the poor quality of the mortar prevent any bond along the thickness and for this reason the overall connection is assumed as very weak.

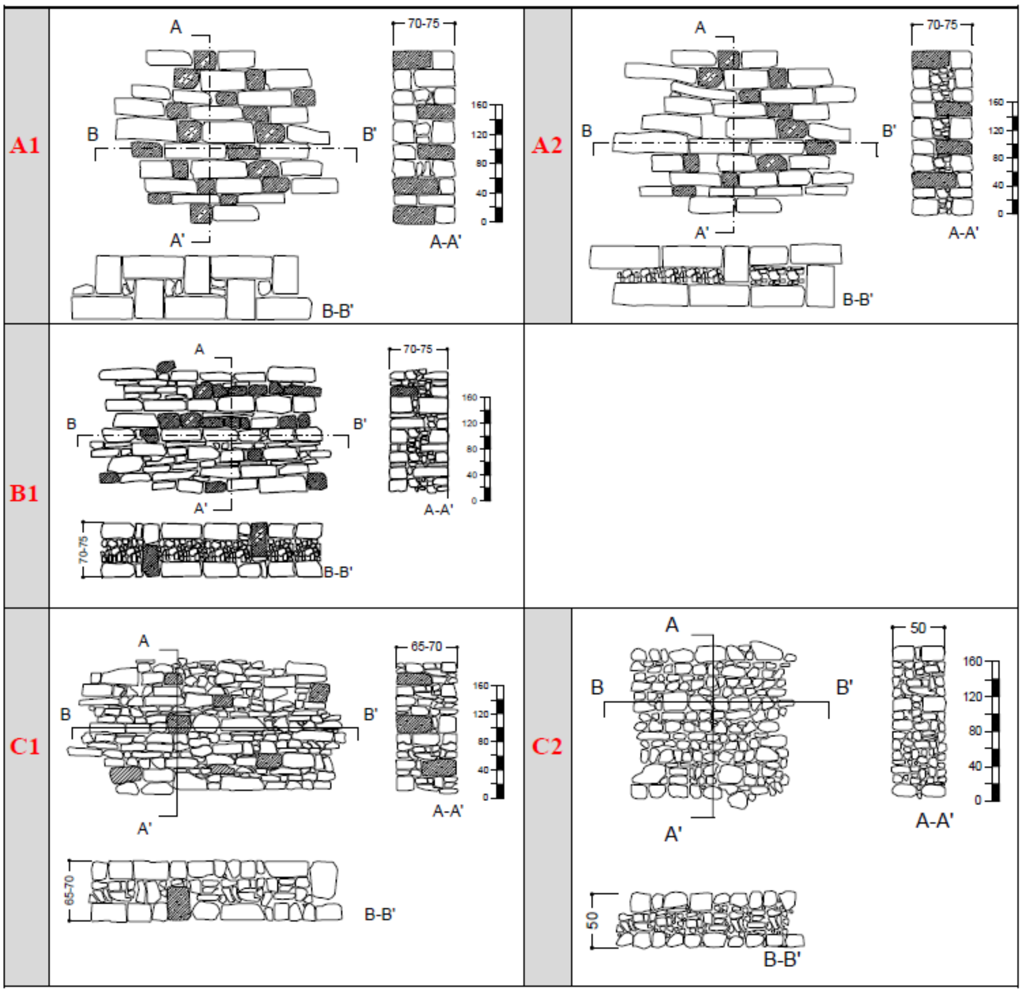


Figure 3.4-Masonry typologies, (Speranza, 2003)

The lack of connection along the thickness of the walls, propitiates their full or partial collapse under the seismic action. Moreover, the quality of the masonry can dictate the buildings performance: the collapse of floors and roofs can be allowed due to the deformability of walls which remain unharmed but provokes the slippage of the horizontal elements out of its supports. However, this is not a common failure based on post-earthquake surveys.

3.1.1.1.1 Masonry Cross-sections

L. Binda and her collaborators, understood the need to characterize and group the various typologies of masonry, based on the first studies of Giuffrè about the mechanical behaviour of the stonework masonry. This operation was easily conducted in those areas where the buildings were damaged by the earthquake and were not repaired by that time, throughout different regions of Italy. First, “an initial cataloguing of multiple leaf walls based on the percentage of mortar, stones and voids measured on the area of the cross section” led to a subsequent “classification based on the number of layers and the constraints between them”.

3.1.1.1.1.1 Brick Masonry Sections

Old brick masonry was usually very thick, with at least 600mm width, and very heterogeneous regarding the brick distribution in the section. In some cases, only the external leaf was made from regular bricks while the inside was made of pieces of bricks and big mortar joints with the purpose of economizing.

Nowadays, modern masonry is made with solid bricks and is classified according its section thickness, number of leaves and connection between leaves.

3.1.1.1.1.2 Stone Masonry Sections

Stone masonry is a traditional form of construction that has been practiced for centuries in regions where stone is locally available. Stone masonry has been used for the construction of some of the most important monuments and structures around the world. Stone masonry buildings can be found in many regions and countries prone to earthquakes. Typically, they are built by building owners themselves or by local builders without any construction techniques. More different typologies are found comparing to brick masonry and are divided in four sub-classes: one leaf solid wall, two leaves, three leaves and dry wall.

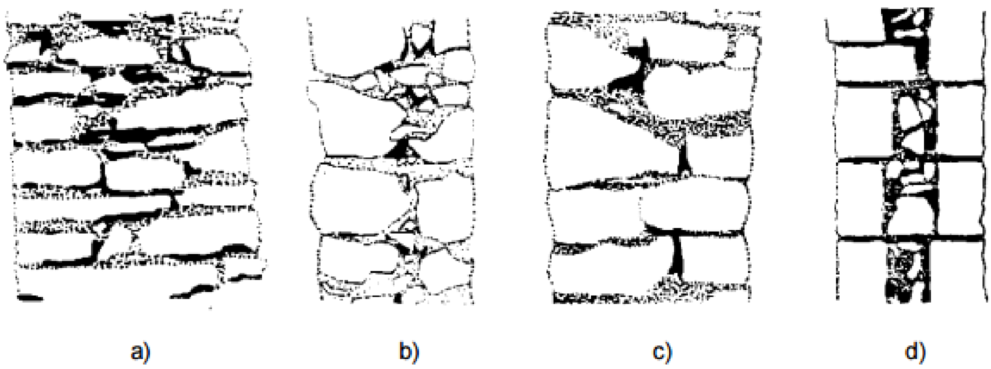


Figure 3.5-Classification of the cross sections: a) a single leaf; b) two leaves without connection; c) two leaves with connection d) three leaves (Binda, 1994)

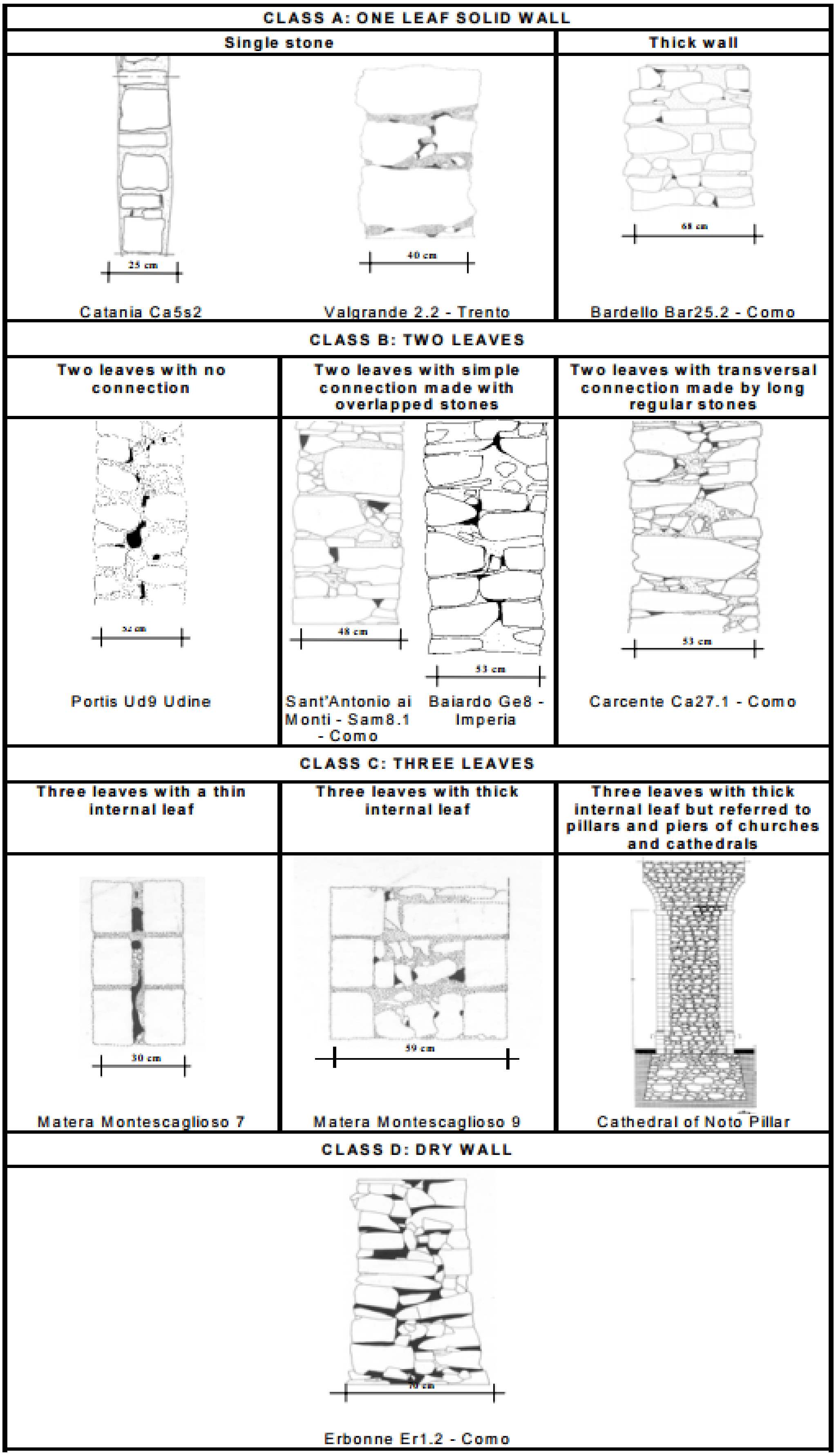


Figure 3.6-Classification of stone masonries, (Binda,2001)

3.1.1.1.2 Masonry Typologies in the Region

Regarding the masonry typologies found in Amatrice and nearby villages, the most regular practice is stones of different sizes randomly placed and connected thought weak mortar (*muratura a sacco*) figure 3.8 b). In Amatrice centre a better masonry was found, such as big square stones in horizontal courses with vertical joints not aligned and, for this reason some buildings survived the earthquake, figure 3.8 a). The third type of stone texture, not really common, is a mixed brick-stone masonry: stone interleaved with brick horizontal courses figure 3.8 c). Brick masonry is also a rare practise and the examples detected were the ones of the following figure 3.8 d).

It is worth mentioning that some textures may seem regular from the outside but, in reality, the interior does not correspond to the exterior. Is this the case of a church in Norcia that collapsed during the 30th October event, figure 3.9 right. The masonry wall corresponds to a three-leaf wall, made by regular big stones in the exterior leaves and, the inside leaf seems limestone. Therefore, a precise investigation of the mechanical behaviour of masonry walls, especially in multi-leaf walls case, is important to understand their seismic performance.

Lateral view of *muratura a sacco* figure 3.9 left where is visible the different stone sizes in a double leaf wall filled with rubble, smaller masonry particles, badly connected to the leaves.

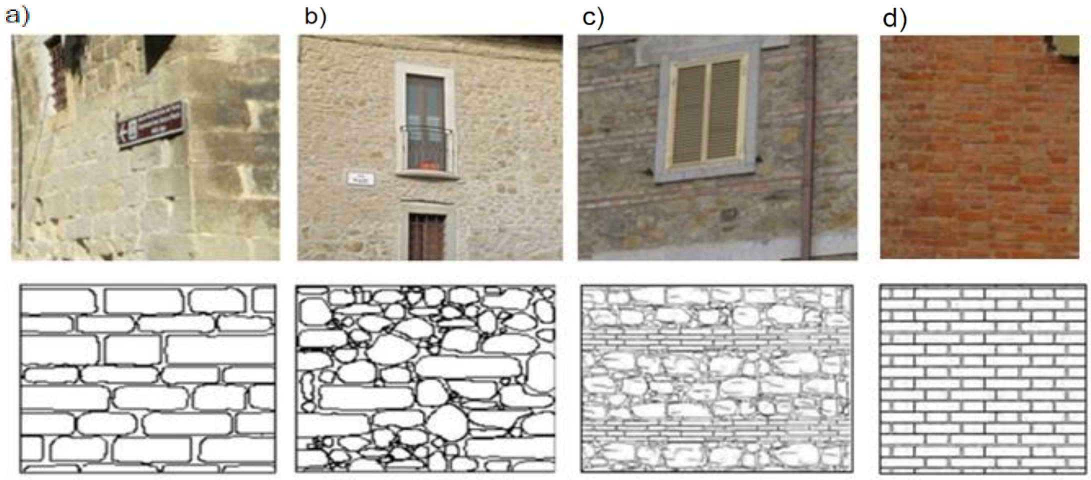


Figure 3.7 -Different types of texture found in the affected area. All the examples are from Amatrice and Pescara del Tronto

Figure 3.8 -From left to right: lateral view of masonry *muratura a sacco* and on the right, Norcia's collapsed church with different masonry in the inside leaf

3.1.2. In Plane Behaviour

Mechanism consisting in diverse typologies of crack patterns associated to individual panels of the masonry walls. In plane behaviour is divided in two distinct failures: flexural behaviour and shear behaviour. On its turn, flexural behaviour is divided in two distinct failures: rocking and crushing figure 3.10 a). Rocking occurs when the pier starts to rotate around its toe. This phenomenon happens when the vertical load is much lower than the compressive strength and the horizontal forces induce tensile flexural cracks at the corners. These cases are likely to happen, for example, in presence of heavy roofs with high horizontal inertial forces. Regarding the phenomenon of crushing, the vertical load is generally high and in the pier, begins to appear vertical cracks in the direction of the more compressed corners.

Shear behaviour also induces two distinct failures: sliding shear failure figure 3.10 b) and diagonal cracking figure 3.10 c). The first consists in sliding on a horizontal plane, generally situated at the ends of the pier. Diagonal cracking is the most common in plane failure and occurs when the masonry walls wore out its capacity of resisting shear forces (tensile stresses exceed the masonry tensile strength which is usually low) and cracks start to appear in the centre of the pier panels and after extending along the panel towards the corners. The cracks can be located both in pier and spandrel panels, figure 3.11, and the cross panels are considered rigid zones. The piers are the principal vertical and horizontal seismic resistant elements; while the spandrels connect the piers in case of seismic loading. These cracks are more likely to appear in the bottom floors where the axial stress is higher. Still regarding diagonal cracking, if the masonry is not good quality the cracks pass mainly through the joints remembering a stair-stepped path.

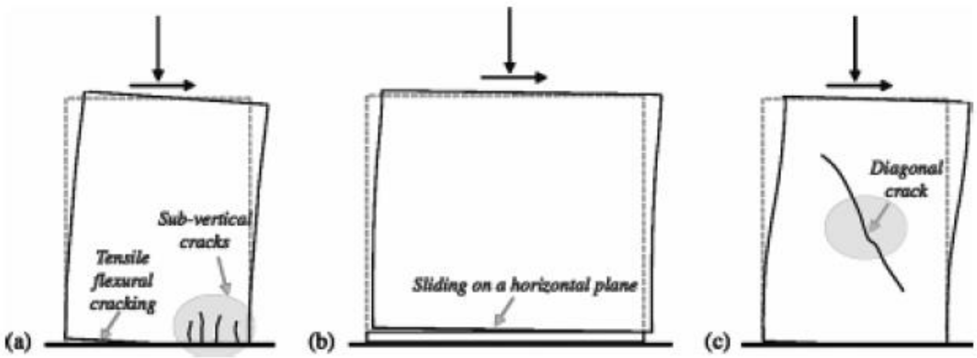


Figure 3.9-Typical failure modes of masonry piers due to horizontal loads: (a) rocking; (b) sliding shear failure; and (c) diagonal cracking, (Calderini, 2009)

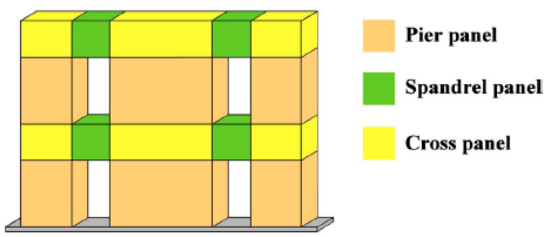


Figure 3.10-Geometrical modeling of a masonry wall

In reality, most of the known cases of in plane mechanism are diagonal cracks. Sliding shear and rocking are not common. Rocking is possible when masonry piers are slender, and when the weight of the structure above is high. Otherwise, the piers are more likely to develop diagonal shear cracking.

3.1.2.1. AeDES Manual

AeDES (Manual for damage detection of ordinary buildings in the post-seismic emergency) was created in 1997 to assess damage, evaluate the post-earthquake usability of buildings and indicate short procedures for damage mitigation. The Umbria-Marche event in that year accelerated its application form. The form permits a fast survey and a first reconnaissance of the building stock, with the gathering of geometrical and typological data of the buildings. Adding these data with damage data, is possible to make a first estimate on the repair and retrofit costs. The form is the result of field experience in numerous past events such as the Irpinia 1980, Abruzzo 1984 and Reggio Emilia 1996. Along the years, it has suffered some modifications. The indications of the following scheme (figure 3.12), should be taken as indicative and effective in case of masonry types which dissipate energy through friction which allows a certain level of resistance after cracks. This is masonry made by solid units, roughly or well dressed, with lime or mixed mortar. Masonry made by hollow units has low structural capacity after the occurrence of cracks. Rubble masonry easily gets damaged and many times has pre-existing damage. When the damage is severe this type of masonry collapses in a brittle manner.

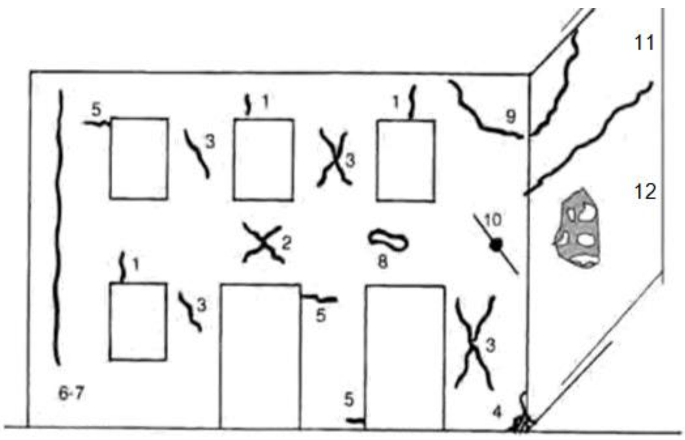


Figure 3.11 -Reference scheme for masonry damage - AeDES

- 1- Nearly vertical cracks on the opening lintels, figure 3.13.
- 2- Diagonal cracks in the spandrel beams (window parapets, lintels), figure 3.14.
- 3- Diagonal cracks in vertical elements (masonry piers), figure 3.15 and 3.16.
- 4- local crushing of masonry with or without material expulsion, figure 3.17.
- 5- Nearly horizontal cracks at the top and/or at the foot of masonry piers, figure 3.18.
- 6- Nearly vertical cracks at walls intersections.
- 7- Same as 6 but with through cracks, figure 3.19.
- 8- Material expulsion at the beam supports due to pounding, figure 3.20.
- 9- Formation of displaced wedges at the intersection of two orthogonal walls, figure 3.21.
- 10- Failure of tie rods or bond slippage, figure 3.22.
- 11- Horizontal cracks at the floor level or at the attic level, figure 3.23.
- 12- Separation of one of the wythes of a double-wythe wall, figure 3.24.

Figure 3.12 -Type 1: the vertical cracks from the two openings are connected, Norcia

Figure 3.13 -Type 2, Arquata del Tronto

Figure 3.14 -Type 3: X shaped cracks between openings, Pescara del Tronto

Figure 3.15 -Type 3: X shaped cracks between the opening and the corner, Arquata del Tronto

Figure 3.16 -Type 4:localized expulsion of material in a church's column due to the overcome of the compression strenght

Figure 3.17 -Type 5, flexural cracks with slight detachment of the plaster on the corners, Norcia

Figure 3.18 -Type 7: vertical cracks along the intersection between buildings evidencing the trigger of out of plane mechanisms, Arquata del Tronto

Figure 3.19-Type 8, Norcia

Figure 3.20 -Type 9, Accumoli

Figure 3.21-Type 10, Accumoli

Figure 3.22 -Type 11: horizontal crack corresponding to the slab roof/floor, Pescara del Tronto

Figure 3.23 -Type 12: localized detachment of the external leaf in the centre of the panel, Norcia

Flexural crack type 5 is relevant and it is probable that in case of aftershocks the damage is much more severe. Regarding the flexural crack type 1, when the damage is restricted the structural risk may be low but, if on the contrary, it is possible that many of the spandrel beams are not able to constrain the masonry piers anymore, the structural risk is high.

Diagonal shear cracking type 2 and 3 are due to the trigger of a shear resistant mechanism inducing the visible displacements. If the displacements are low and restricted, the structural risk is declared low. However, if the displacements are significant the risk for the building is high. In figure 3.16, shear cracking in the corner is close to the partial collapse. Often, this typology of cracking leads to the trigger of a complex mechanism, including out of plane bending of the walls panels. The bending is well visible in these cases and the partial collapse is likely to happen.

Cracking of type 4, might be an indication of crushing failures. The performance of the masonry towards this failure mechanism is usually brittle, especially in solid brick masonry and hollow brick masonry. The risk which this type of cracking carts depends on the wall geometry and typology and extension of the damage, which specifies how much the vertical bearing resistance is affected. If a high concentration vertical stresses happened (result from the presence of openings that diminishes the load bearing system, for example) the structural risk is declared high, mainly in buildings with poor quality masonry and lack of maintenance.

Vertical cracks in the intersections between orthogonal walls, type 7, are the proof of lack of connections between the referred walls and the original structural configuration has been changed. Cases like these need careful attention when evaluating the structural risk, because in case of aftershocks the activation of an out of plane mechanism is probable. Type 6 is similar to type 7 only with the difference that in this case the cracking is not through the masonry wall.

Type 8 cracking should be declared as high structural risk if there is a decrease of the load bearing capacity, usually connected to out of plumb result of pounding actions. In the picture, this failure occurred allied to others more complex.

Type 9 cracks are clearly detected when there is the separation of the wedges of the masonry building. If this separation involves significant displacements the structural risk is high.

Type 10 failure consists in tie failures or bond slippage with out of plumb related. The high structural risk of tie failures is associated to the consequences on the masonry building the modification of the static configuration carts.

Indications of out of plumb are usually connected to cracks type 6 and type 7, with probable disaggregation between the walls and floors, and the structural risk generally associated is high. When this phenomenon is visually evident it is necessary attention: if the masonry wall configuration is a double-leaf (section 3.1.1.2) or *sacco* (section 3.1.1.3) it is likely to occur separation between leaves. In these cases, type 12 cracks appear.

A type 11 crack with displacements of mm is the sign of sliding between the floors level and the masonry wall underneath. The most common case of sliding is between the roof and the masonry walls. This effect is due to the thrusting roofs and it is possible to cart a high structural risk, result of the combination between thrusting roofs and the alteration introduced by the sliding.

3.1.3. Out of Plane Behaviour

The first damage mode, out of plane, is based on the overturning of walls, subjected to forces in the perpendicular direction, and is divided in four main fields: simple and composite overturning, vertical and horizontal bending. This subject is well documented in the literature and the conclusion of several stone masonry buildings damage in earthquakes is that the most recurrent failure mechanism surveyed is the overturning of the street façade. This mechanism can involve the entire wall or just a portion of it. The way in which this will develop depends on the quality and strength of the connections between the elements of the structure (structural walls, partition walls, floors and roof).

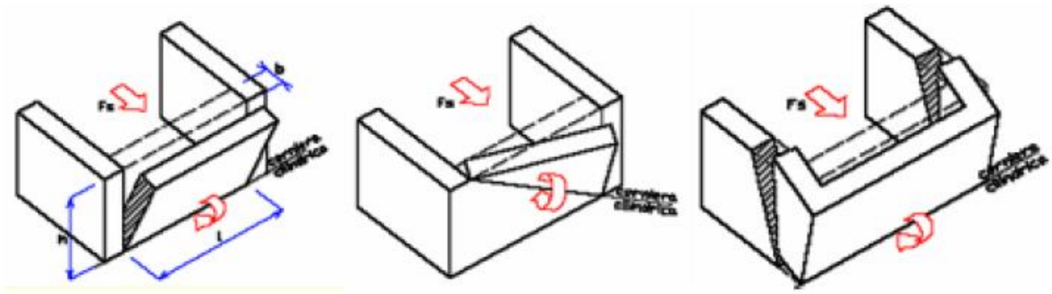


Figure 3.24-Overturning mechanisms. From left to right: simple overturning, partial overturning and composite overturning (Borri, 2004c)

However, if the buildings were subjected to interventions to improve its seismic behaviour, the simple overturning is precluded and mechanisms on arch effect appear (horizontal and vertical bending):

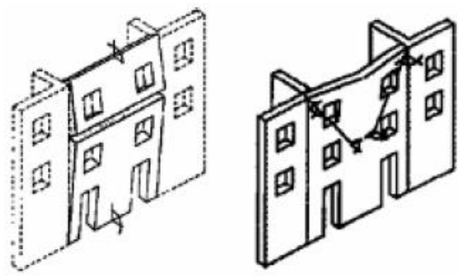


Figure 3.25 -From left to right: vertical bending and horizontal bending

The following sub-chapters aim at giving a brief theoretical explanation on out of plane mechanisms and associating examples from the 24th earthquake. The approach chosen is based on a failure analysis of the structures through the identification of suitable collapse mechanisms (figure 3.27). This methodology of quantifying the seismic vulnerability has been applied in the description and definition of damage scenarios, for example in L'Aquila in 2009.

The formulation proposed by the authors, D'Ayala, Speranza and Novelli, includes eight out of plane failures and collapse of roofs and tympanums which are also treated as out of plane collapses. As the analysis is conducted storey by storey, it is also possible to identify the number of floors involved.

In plane failures are also indicated but with much less detail than in section 3.1.2.1., only including diagonal cracking of masonry façades.

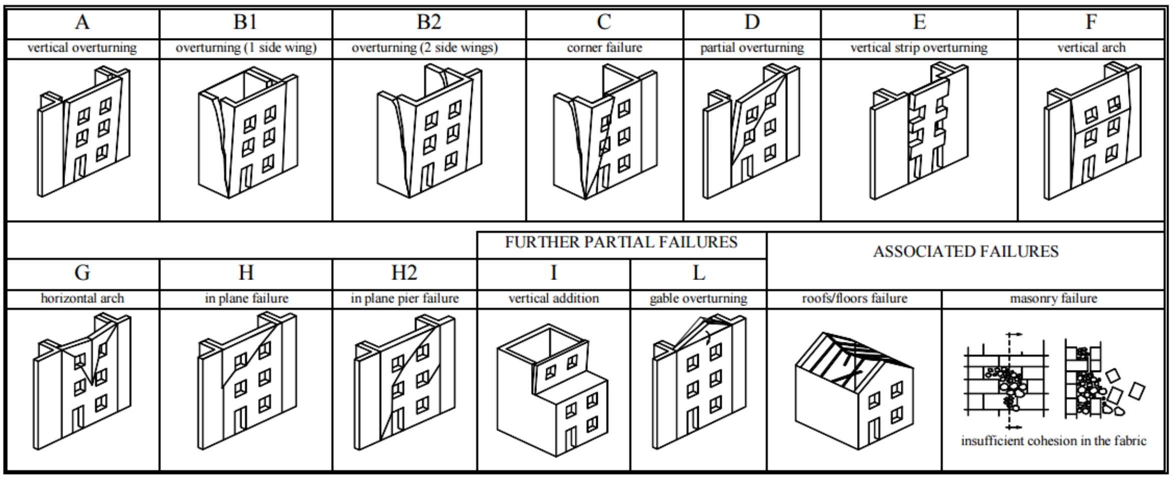


Figure 3.26 -Façade mechanisms of failure, (D'Ayala and Speranza 2002.2003; D'Ayala and Novelli 2011)

3.1.3.1. Simple Overturning

This mechanism is manifested through the rigid rotation of the entire façade, or portions of walls, around its horizontal axis in the base, due to acting forces in the perpendicular plan. Symptoms manifested by the damaged wall are vertical cracks along the intersection between orthogonal walls, slight inclination of the wall to the direction of its less stiffness (out of plumb) and pull out of the beams from the slabs. In the worst situation, the wall is free on top, without any restrain, and not connected to the lateral orthogonal walls. Different masonry portions, local damage and opening position/geometry could start the overturning.

Weaknesses and vulnerabilities associated to simple overturning can be deformable diaphragms or diaphragms not well connected with the vertical elements, absence of ring beams or ties beams, poor quality of intersections between walls or poor quality of masonry. Overturning of lateral walls can also be because of thrusting roofs. The overturning mechanism can have different variants, involving one floor or the whole façade, as aforementioned, depending on its connections to the slabs, the entire thickness of the wall or the outer face only and distinct geometries on the wall due to the presence of discontinuities or openings.

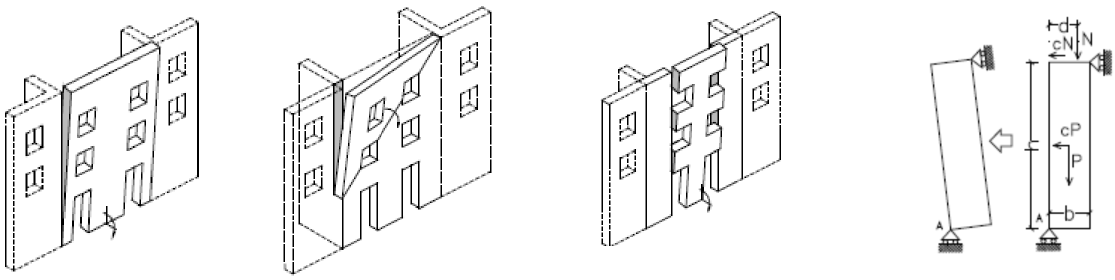


Figure 3.27 -Possible mechanisms of collapse for simple overturning (D'Ayala, 2003a.). From left to right: vertical overturning, partial overturning and vertical strip overturning. The latter occurs when vertical cracks causer the detachment of a façade strip, due to the different stiffness between openings and piers. On the right, the scheme shows the ooverturning of a wall simply supported by the orthogonal wall

Figure 3.28 -Accumoli From left to right: vertical overturning, partial overturning and vertical cracks along the intersection between orthogonal walls

3.1.3.2. Composite Overturning

This mechanism follows the same principles as the last one, only with the peculiarity that the bracing walls are also dragged, demonstrating an effective connection between orthogonal walls, unlike simple overturning. The symptoms are similar to simple overturning and vulnerabilities associated are absence of ring beams or tie beams, deformable diaphragms or diaphragms not well connected with the vertical elements, presence of thrusts not supported by walls, presence of openings in the vicinity of the walls intersections which influence the structural behaviour during the earthquake and poor quality of the masonry.

In case of walls without openings the inclination of the diagonal crack increases with the masonry quality corners.

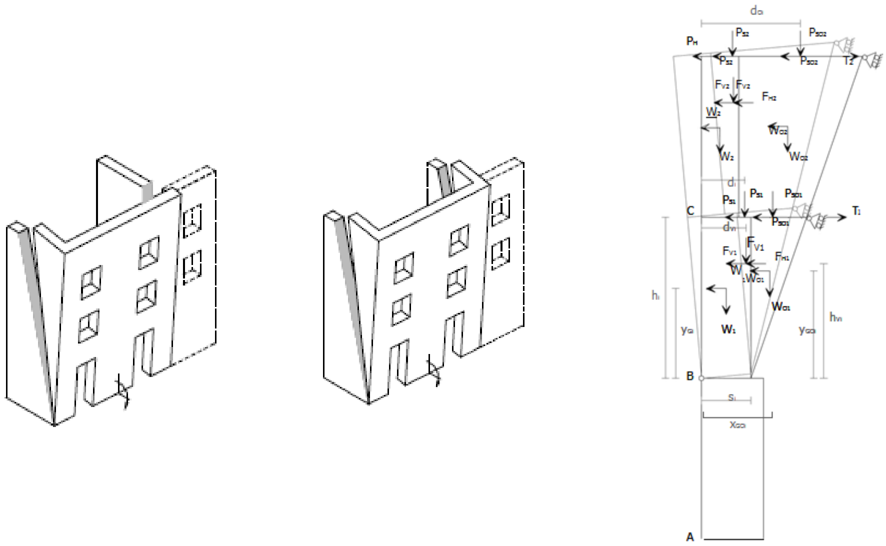


Figure 3.29 -Development of overturning mechanisms for coupled walls (D'Ayala, 2003a). From left to right: overturning (one side wing) and overturning (two side wings). On the right, kinematic mechanism, RELUIS

Figure 3.30 -Composite overturning (one side wing) in Pescara del Tronto and Amatrice

3.1.3.3. Corner Overturning

Corner overturning consists in the rigid rotation of a detached wedge around a hinge in its base. The fracture surface is triangular, characterized by two diagonals as shown in the figure 3.32. Mechanisms of this type are common in buildings that have high thrusts on the upper corners due to the loads transmitted by the struts of hip roofs. It is assumed that the overturning occurs in the direction of the strut thrust and that the collapse surface forms an angle of 45 ° with the converging walls.

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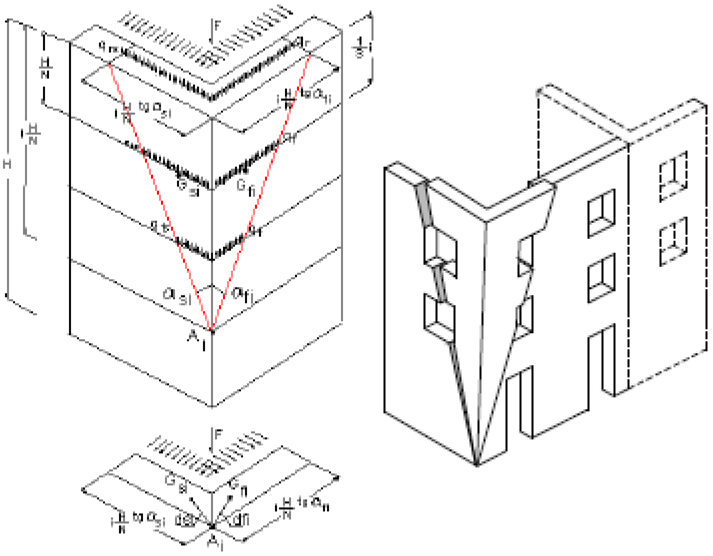


Figure 3.31 -One free corner in an isolated building or end/corner building in a row/group

Figure 3.32 -From left to right: corner failure, probably exacerbated by the presence of a chimney which contributted to the high thrusts coming from the roof, Accumoli. On the right: Corner failure in a hip roof, demonstrating the triangular collapse surface, Accumoli

3.1.3.4. Vertical Bending

The mechanism is characterized by the rotation of the wall, divided in two blocks, around a horizontal hinge in the presence of out of plane actions, due to the introduction of ties beams or ring beams, as aforementioned. The upper and the lower bonds, in general, are effective to prevent the global overturning of the wall. On the contrary, orthogonal walls have disable connections allowing the wall bending.

Buildings conducive to this failure mechanism are characterized by excessively slender walls, poor connection between diaphragms and vertical elements, poor connection between leaves and high horizontal thrust due to the presence of arches or vaults.

Symptoms associated with the activation of this mechanism are relevant out of plumb of the damaged wall, horizontal and vertical cracks and extraction of the beams from the slabs. Regarding the different variants of vertical bending, this mechanism can involve one or more floors, depending on the connections between horizontal and vertical elements, the entire thickness of the wall or the outer facing only, in relation to the characteristics of the wall structure and distinct geometries of the collapse surface depending on the presence of openings or discontinuities.

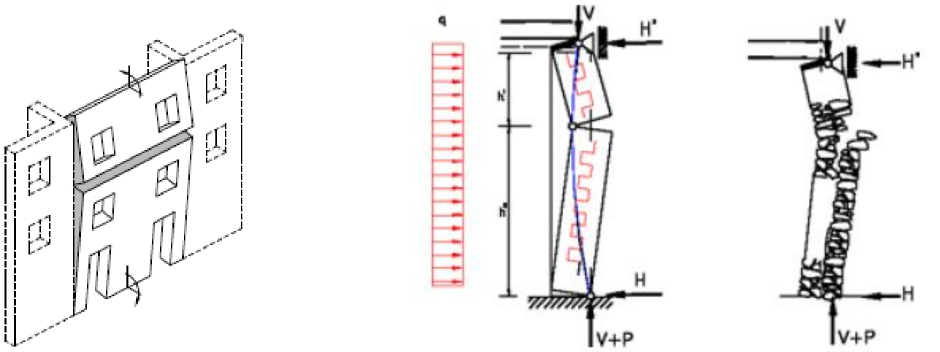


Figure 3.33 -On the left,vertical arch mechanism due to presence of ties or ring beams at the top of the wall. On the right, development of vertical arch mechanism (D'Ayala, 2003a)

Figure 3.34 -From left to right: vertical bending involving one floor, vertical bending involving two floors, Arquata de l Tronto and relevant out of plumb showing signs of an incipient vertical bending, Amatrice

3.1.3.5. Horizontal Bending

The mechanism occurs with the expulsion of material from the summit area of the wall, through an arch mechanism, caused by the out of plane actions. The arch mechanism is characterized by the development of three hinges, one in the middle of the wall section and the others close to the connection to the lateral walls. It takes place when there is an effective linking between the orthogonal walls and no constraints on the upper part of the building. The mechanism is typical of walls restrained by tie rods. The beam hammering or the roof thrust and the low quality of the masonry could produce the whole mechanism or local damage.

Buildings vulnerable to this type of mechanism present defective connections between roof and masonry walls, openings or discontinuities that reduce the load bearing section, slender walls and thrusting roofs *(coperture spingenti).* Symptoms manifesting the successful activation of the mechanism are vertical and oblique cracks on the external and internal face of the wall, presence of swellings and extraction of the roof beams.

Similar to vertical bending, this mechanism has distinct geometries of the collapse surface depending on the presence of openings or discontinuities and in multiple leaf masonry the collapse can be total or just the external leaf.

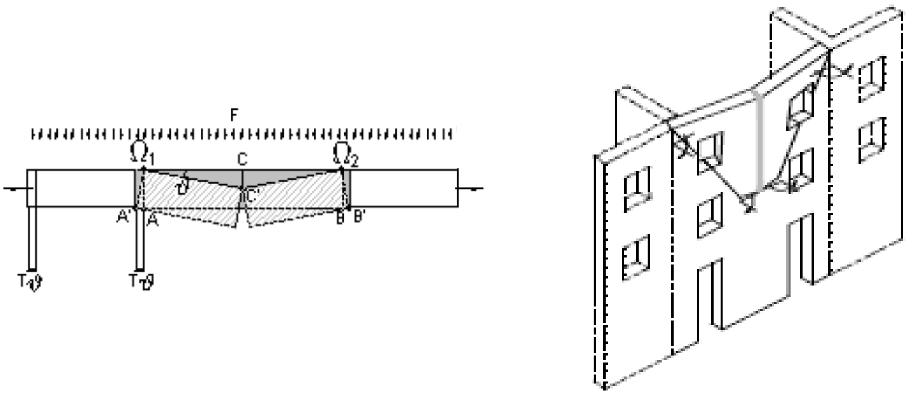


Figure 3.35 -Development of horizontal arch mechanism (D'Ayala, 2003a)

The concept *coperture spingenti*, thrusting roofs, can be easily understood with the following figures. In the absence of ties beams, lateral displacements of the load bearing walls occur due to the horizontal thrusts coming from the roof.

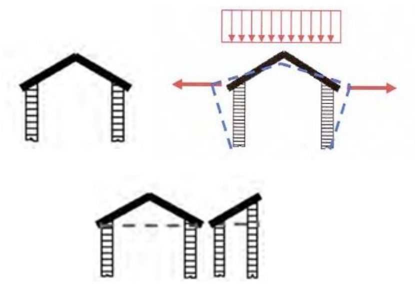


Figure 3.36- On top: *Coperture spingenti*. On bottom: *Coperture non spingenti* (source AeDES)

Figure 3.37- Evident out of plumb reveling the horizontal bending, Scai, Amatrice. The tie beams visible in the picture were not capable of avoiding the triggering of the mechanism

3.1.3.5.1 Tympanum Collapse

This case is a particular case of horizontal bending because it deals specifically with the tympanum collapse of buildings due to the hammering of the ridge beam. During the earthquake action, the presence of ridge beams provokes the transference of thrusts to perpendicular wall tympanums causing their collapse.

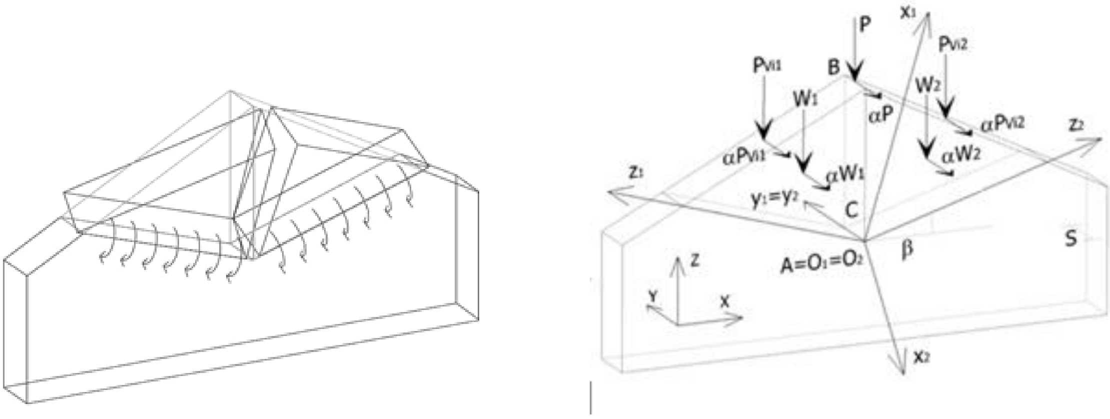


Figure 3.38 -Schematic and kinematic model of a tympanum collape

Figure 3.39-From left to right: tympanum collapse of a church and start of its reconstruction. On the right, tympanum collapse of a residential building, Norcia

3.1.3.6 Roof Failure

Roof collapse is common during the occurrence of earthquakes and is one of the most fatal mechanisms. It can take place when either the walls lose the ability to resist gravity loads and collapse, or when the roof structure collapses. The latter case is often caused by inadequate wall-to-roof connection: the roof structure can simply move out from the walls and cave into the building.

Figure 3.40-Roof collapse, from left to right: Pescara del Tronto and Arquata del Tronto

Figure 3.41-Schematic model of roof collapse

3.1.4. Pounding

The need to build adjacent buildings forces the designers to take under consideration earthquake-induced interaction between buildings. Interaction between buildings results from adjacent buildings with different heights under dynamic loads: the lower building damaged the taller building by hammering against non-structural elements. Hammering, between adjacent cells could also lead to the overturning of the smallest adjacent volume. For this reason, buildings should have the same height. This mechanism occurs both in masonry structures and reinforced concrete structures.

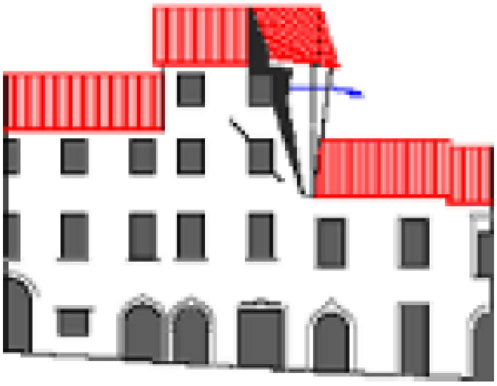


Figure 3.42 -From left to right: Schematic model of an overturning wall due to hammering and a real case of pounding in Arquata del Tronto, 2016

Figure 3.43 -In this case the overturning of the wall does not occur, only cracks in the intersection between the two buildings, Amatrice, 2016

3.1.5. Bell Tower Failure

Two bell towers, one in Arquata del Tronto and one in Norcia (30th October event), suffered rotation and sliding of the bell piers, with local corner expulsion as the scheme explains.

Figure 3.44-From left to right: Arquata del Tronto and Norcia

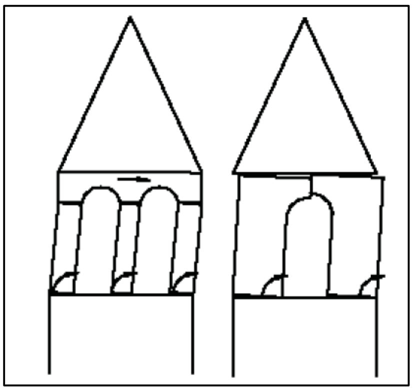


Figure 3.45-Schematic model of the piers rotation and sliding

3.2. REINFORCED CONCRETE STRUCTURES

Damage in reinforced concrete buildings can occur in structural elements and non-structural elements. Structural elements are those considered part of the structural system and are divided into two groups: primary and secondary elements. Primary elements resist the seismic loads and are designed according to the regulatory standards. Secondary elements are only designed to support gravity loads when subjected to the displacements caused by earthquakes, so they are not part of the resisting system (e.g. stairs).

Non-structural elements are not considered load bearing elements in the building design (e.g. infill walls). Although infill walls drastically modify the structural response, attracting forces to parts of the structure which were not designed to resist them, due to the significant increase of global stiffness in the structure, they are not considered in the system design. Even though this seems contradictory, infills are very complex structural elements due to its non-linear behaviour, hence the modelling is highly complicated.

Reinforcing steel also plays a significant role in the seismic response of a reinforced concrete structure therefore, for regions of moderate to high seismic risk it is necessary to carefully design the detailing (joints, lap-slices, anchorages, stirrups/hoops) and the proper amount of reinforcement. Varum (2003) indicates the most common causes of failure or damage in reinforced concrete buildings: lack of stirrups/hoops, confinement and ductility, bond/anchorage/lap-slices slipping and bond splitting, inadequate shear capacity, inadequate flexural capacity, inadequate shear strength of the joints, influence of the infill masonry on the seismic behaviour of frames and strong-beam weak-column mechanism.

Each of the next sub-chapters explains these failures and illustrates, every time possible, the mechanisms with examples from the 24th August earthquake. In some cases, such as, inadequate flexural capacity, short column and strong beam weak column, this methodology was not followed due to the lack of examples in the seismic event from last year. Pictures from other earthquakes are shown instead, including the Turkish seismic events on 1999 and 2011 and finally the 1994 event in United States of America.

3.2.1. INFLUENCE OF THE INFILL MASONRY ON THE SEISMIC BEHAVIOUR

The contribution of non-structural elements in the design of new buildings is typically not considered, as well as in the assessment of existing ones. In the case of the infill masonry walls, two principal mechanisms have been often observed after the occurrence of earthquakes: the soft-storey mechanism and the short column mechanism.

3.2.1.1. Soft-Storey Mechanism

A very common irregularity in buildings appears at the lower storey levels, resulting from the absence of infills, contrary to the upper storeys. This characteristic is especially seen in buildings with commercial purposes at the lowest storeys and offices or residential uses above, leading to large clear spaces below unlike upper floors with significant portion of the total stiffness due to infills. Thus, the resulting structural frame system is irregular and, if not accurately designed, inadequate to resist earthquakes.

Other important characteristics towards the seismic behaviour are the vertical regularity (all the columns must have the same height) and horizontal regularity (maintain the rigid centre close to the mass centre to avoid torsion).

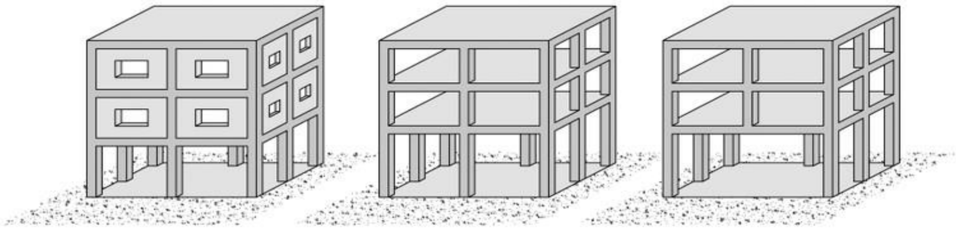


Figure 3.46-From left to right: stiff and strong upper floors due to masonry infills; columns in one storey longer than those above and soft storey caused by discontinuous column

Figure 3.47-From left to right: evident soft storey mechanism of the Hotel Roma, Amatrice. The lack of stirrups and the use of smooth bars facilitated the mechanism. On the right, incipient soft storey mechanism revealed by the separation between the concrete column. Also the coatings buckling reveales this mechanism. In the joint is visible lack of transverse reinforcement, Norcia

Figure 3.48 -In this case the soft storey ocorrued in a masonry building: the first floor completely disappeared and the second and third floors remained standing, Norcia

3.2.1.2. Short Column Mechanism

Columns are shortened by elements like infills, openings, and stairs, which were not considered during the design. As the example presented below shows, the infill walls cause the plastic hinges formation on top of the infills and on top of the columns, leading to shear failure of the columns. The term short column is due to the fact that the actual length of the columns is the one that is not constrained by the infill walls, hence the actual length resisting the seismic load is much lower than the expected, causing columns shear failure.

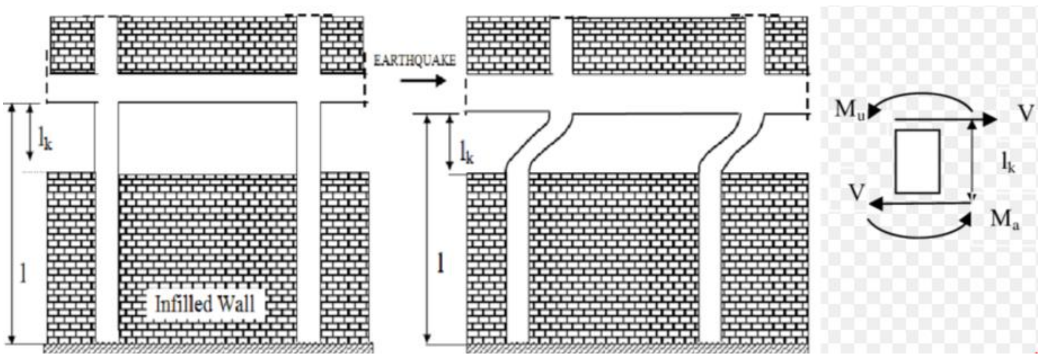
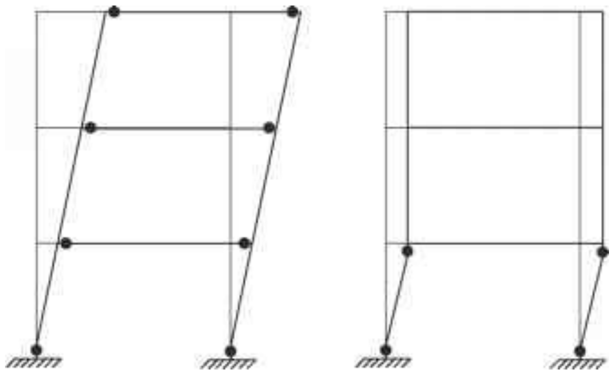


Figure 3.49-Partial masonry infill in concrete frame

Figure 3.50-Columns shortened by the presence of openings with visible X-shaped shear cracking. From left to right: Turkey, 2011 and United States of America, 1994

3.2.2. Strong Beam Weak Column Mechanism

In recent earthquakes, many reinforced concrete structures have collapsed or were severely damaged due to the development of the strong-beam weak-column mechanism. It occurs when the formation of the plastic hinges starts in the columns and not in the beams extremities, which is the most desirable location, in order to design a ductile and capable of dissipating energy structure. Eurocode 8 suggests that columns resisting moment should be 30% higher than the beams resisting moment.



$$\sum M_{R,column} \geq 1.3 \cdot \sum M_{R,beam}$$

Figure 3.51-Schematic model demonstrating the location of plastic hinges. In the left the plastic hinges appear in the beams extremities unlike the second case where it appears in the end of columns. To prevent this mechanism the safety factor must not be less than 1.3

Regarding the figure 3.53, in the upper line of sketches the building has thick and stiff floors with slender supporting columns. During the earthquake, the bottom columns receive the largest forces and bend; walls crack and the whole building will fail. In the second line of sketches the floors have a ductile design, allowing the absorption of some of the shock. Floors will be waving and cracking and with properly designed columns the façade may crack, but the building will not collapse.

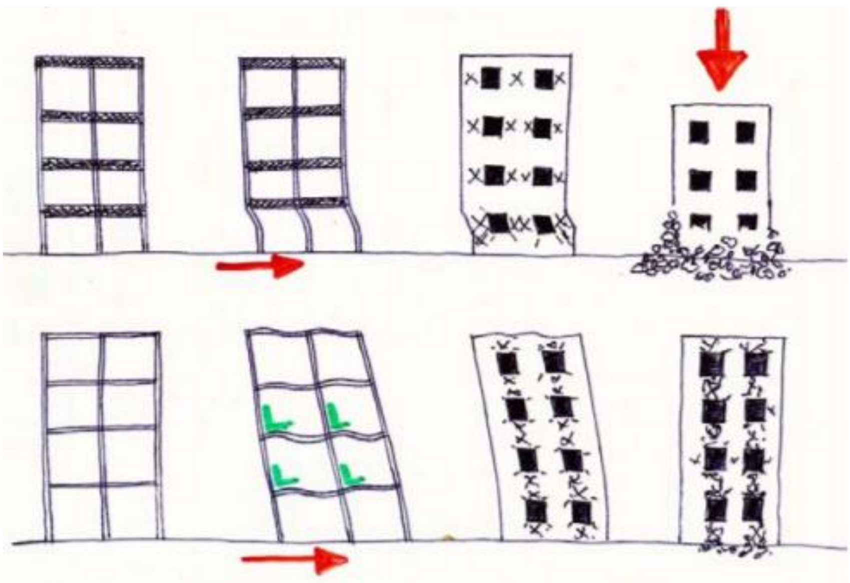


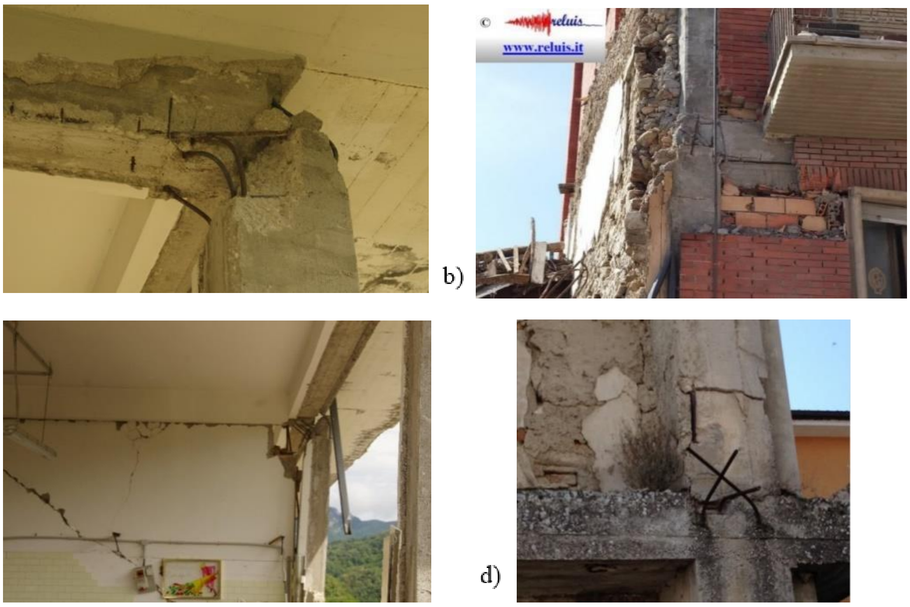
Figure 3.52-Different seismic behaviour of the two multi-storey structures

Figure 3.53-Plastic hinge formed in the beam column joint. From the observation, it is obvious the fragility of the column compared with the monumental slab it was supporting, Pakistan, 2005

3.2.3. Lack of Stirrups/Hoops, Confinement and Ductility

Many structures exhibit deficient confinement due to inadequate transverse reinforcement, leading to brittle collapses of the compressed columns. Adequate detailing and amount of stirrups, delays the collapse by increasing their strength and ductility. This confinement depends on several parameters such as the stirrups diameter, stirrups spacing, steel quality and on the shape of the stirrups and cross-section. Regarding the figure 3.55, in the cases a), b) and c) the lack of detailing is concentrated in the beam-column joint while, in the picture d), is visible the absence of stirrups in the column.

When it comes to the beams, local failures do not lead necessarily to collapse, unlike column case. Beams and beams-column failures are usually connected to inadequate transverse reinforcement in the zone of plastic hinges formation. In short words, proper quantity and detailing, regarding the required ductility, of plastic hinging zones is fundamental to a good seismic behaviour.



a) b) c) d)

Figure 3.54-Damage in reinforced concrete beam column joint due to lack transverse reinforcement in a), b) and c). a): absence of stirrups/hoops in the beam column joint and evident poor quality of the concrete, Pescara del Tronto. b): clear separation between column and beam in the joint due to absence of reinforcement in the joint, Amatrice. c): absence of stirrups in the beam column joint, Pescara del Tronto. d): lack of stirrups in the column, Amatrice

3.2.4. Bond/Anchorage/Lap-slices slipping and Bond Splitting

Bond in reinforced concrete refers to the resistance of surrounding concrete against the pulling out of reinforcing bars, and depends on factors such as type of reinforcing steel and stress state in both reinforcement and surrounding concrete. High-adherence bars are important to enhance interlocking and "smooth bars" should be avoided. Also, these characteristics play an important role: concrete cover, space between reinforcing bars and bars position. Bond is necessary not only to ensure an adequate level of safety allowing composite action between steel and concrete, but also to guarantee a ductile behaviour, especially in case of dynamic loads. Bond failure results from splitting of the concrete cover surrounding the reinforcing steel and is typical of elements in which reinforcement is anchored with minimal concrete cover in a region with a minimal volume of transverse reinforcement.

The transverse reinforcement surrounding lap slices, either in the form of stirrups or hoops, can improve their behaviour and strength, delaying the splitting of concrete. This and more rules should be applied in the use of lap-slices and anchorage such as, avoiding the plastic hinging zones to do lap-slices and anchorages and avoid the use of big bars diameter because the anchorage length increases with its bar diameter.

Figure 3.55-Inadequate lap-slices and lack of stirrups in both figures. From left to right: lap-slices done in a sensitive zone and plane reinforcement bars diminishing the friction between them and concrete, Italy 2009. On the right: lap-slices appear in the end of columns, the zone of the first plastic hinges, Turkey, 1999

Figure 3.56.1-In this case of the hospital in Amatrice the anchorage of the steel reinforcement seems to have been done before the end of the pier

3.2.5. Inadequate Shear Capacity

Inadequate transversal reinforcement in terms of size, spacing and detailing is the principal cause of shear failure. Also, insufficient area of transversal reinforcing steel, with large spacing and deficiently anchored to the concrete core is very common. Visually, this failure is detected by diagonal fractures in the columns as shown in the figures.

Structures reach the brittle collapse by columns shear failure before the yielding of the reinforcing steel bars, dictating the bad capacity design and the lack of ductility in the system. Earthquakes in the last decades have been showing the major importance of ductility, the capacity of buildings to dissipate energy and reach the collapse with sufficient advance to evacuate the building. Nowadays, these characteristics are fundamental in the design of infrastructures.

In some cases, it may be difficult to distinguish flexural compression and shear compression failure, as both occur in or near the column ends and involve crushing. The corner columns are the most critic ones, especially if the structure is irregular in the plan, what leads to torsion, hence these columns need to have a stronger confinement.

In order to increase the shear capacity, it is mandatory an adequate quantity of stirrups and ties to well confine the concrete, use a good concrete quality and avoid shear and tension simultaneously in the column.

Figure 3.57-Shear failure of a corner column in a five-storey building accompanied by damage in the beam-column joint. The buckling of the longitudinal reinforcement is visible. Once again, the importance of detailing, bars anchorage and shear strength, Amatrice

Figure 3.58 -Shear failure of the concrete column due to the shear action of the masonry panels, Norcia

3.2.6. Inadequate Flexural Capacity

Many brittle failures are observed, as well, in flexure, especially in older buildings, when the capacity design rarely included ductility in structures. To achieve a ductile building, measures such as limiting the compression axial forces or increasing the cross section area, using good quality concrete, adequate confinement and limiting the area of reinforcement steel bars, because its area is proportional to its yield stress, can be taken.

In some cases, shear and flexure appear simultaneously, typically in more slender columns. These columns have higher shear strength than flexural strength, which allows them to yield in flexure prior to shear failure.

Figure 3.59-Flexural deficient behaviour in reinforced concrete columns. On top San Salvatore Hospital, Italy, 2009. On the left, front view and on the right, back view.

3.2.7. Inadequate Shear Strength of the Joints

The connections between structural members are fundamental for a satisfactory seismic behaviour, otherwise the stiffness and ductility of the elements are not even put to the test. Defective connections are those which have improper detailing of transverse reinforcement and confinement, inadequate shear strength and inadequate anchorage capacity.

Figure 3.60-Inadequate joint behaviour in residential buildings. a) damage in the beam column joint with expulsion of the concrete cover due to lack of stirrups, Pescara del Tronto. b) damage in the joint connecting two columns and one beam - *ginocchio*, Hotel Roma, Amatrice. c) crisis in the joint between the column and the ring beam at the floor level, Amatrice. d) crisis in the joint due to lack of stirrups, Amatrice

3.2.8. DAMAGE IN STRUCTURAL SECONDARY ELEMENTS

Secondary elements are not considered during the seismic design, as aforementioned. However , these elements, which examples can be cantilievers and stairs, have a positive contribution when subjected to earthquake actions in the plan of their large stifness, acting as stabilizer elements. If, on the contrary, they are incorrectly positioned in the horizontal plan or are not well connected to the rest of the structural system, the seismic response is agravated. During the seismic event the behaviour of these elements is impredictable because the global model does not include it. Damage to stairs is typically due to its bracing performance in the framed system. Therefore, the connection between the frame and the stairs needs special attention, for instance, at the connection between the stair beam and the column (figure 3.63). Diagonal forces may cause a horizontal load on the middle of a column, creating a moment force to which the column was not designed, short column. Stair behaviour is also determined by the lack of reinforcement detailing and occurrence of damage in the beams extremities supporting the stair slabs .

Cantilievers usually suffer excessive deformation and stairs in the lower floors are mainly the more damaged.

Figure 3.61 -From left to right: excessive deformation in a cantilever, also due to the vertical seismic component. On the right, total collapse of the stais in the first floor. Spain 2011

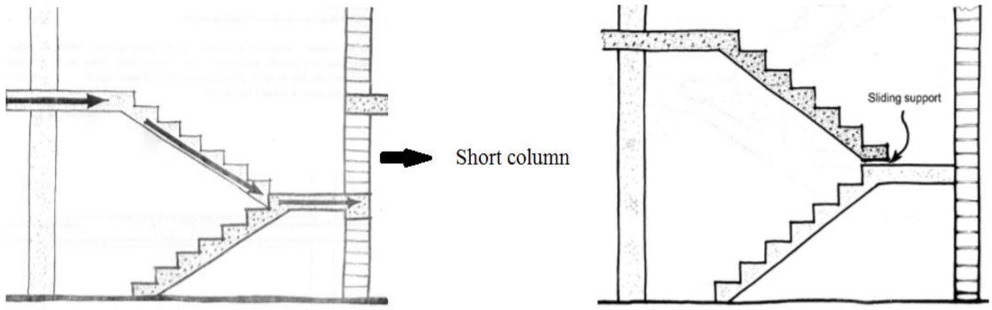


Figure 3.62 -From left to right: short column created by a stair landing and on the right, sliding support in the connection between the two stringers

Figure 3.63 -From left to right: in this case is visible a large crack between the half pace and the last step. The stairs typology is clearly cantilever steps, Italy 2009. On the right, there is also a deep crack in the connection between the landing and the stringer probably due to lack of reinforcement detailing, Arquata del Tronto

3.2.9. Damage in Non-structural Elements

Damage in elements such as infill walls, roofs or chimneys, are very common in reinforced concrete structures and easily detectable to the naked eye. Infill walls can be used as interior partitions and as external walls in concrete frames and are typically made of brittle materials that lose capacity in a rapid way. The presence of masonry infills can have positive and negative effects in the whole building. When it is positive, if effectively confined by the frame, infills are remarkable in increasing the initial stiffness, strength and energy dissipation of reinforced concrete frames. If the effect is negative, causes a significant increase in the demand forces on the diaphragm which results in brittle shear failures (soft storey), short column phenomena, and torsional response to the horizontal components of the seismic action. The structural response of infilled frames depends on numerous parameters. Overall geometry of infills, dimensions of concrete members, the diversity of mechanical properties of infill and concrete members, reinforcement configurations, location and dimension of openings, distribution of masonry infills walls throughout the storey and construction details are some of these important parameters. Although, infilled frame buildings are well documented in the literature, there are still a lot of uncertainties regarding the interaction between the frame and the infills. This interaction can change the seismic response significantly.

As already studied in sections 3.1.2 and 3.1.3, damage in infill walls can be divided in plane and out of plane failure. Generally, in plane behaviour is responsible for the formation of diagonal cracks and separation between infills and the main resistant structure. Out of plane behaviour consists in infills walls overturning: exterior leaf or entire wall overturning. Parallel to these cases exists coating overturning that results from poor connections to the panels. Parapet is another non-structural element usually underestimated during the design although its response to dynamic loads is dangerous. Often parapets are not well connected to the main structure and consequently fall over the streets causing many damage.

3.2.9.1. Infill Masonry In Plane Failure

The most common failures modes in infill masonry is separation of the infill from the structural system, (figure 3.66), diagonal cracks (figure 3.65) and/or displacements of few mm (figure 3.65 right) and evident crushing at the infills corners. In some cases, the detachment of material also occurs (figure 3.65 left). This damage in infill masonry demonstrates its considerable contribution to the building behaviour

Figure 3.64-Damage in interior walls .From left to right: large diagonal crack in the infill wall with separation from the main structure. On the right, X-shaped crack forming a “stair-stepped’ path due to the masonry’s weak adhesion, Norcia

under seismic actions. However, in case of aftershocks, the infills would not be able to response the same way.

Figure 3.65- Damage in exterior walls. From left to right: clear separation between the infill walls and the structural system, Norcia. On the right, precendent case plus X-shaped cracks between two openings, Arquata del Tronto

3.2.9.2. Infill Masonry Out of Plane failure

One of the major factors that causes the out of plane instability and poor performance of the building is the deficient and/or insufficient support-width on the reinforced concrete beams and/or slabs, normally adopted to minimize the thermal bridges effect, no connection between the interior and the exterior panel leading to collapse of exterior panels, and finally no connection to the surrounding reinforced concrete frames.

Figure 3.66 -From left to right: collapse of infills and collapse of external infill, Norcia. On the right, out of plane mechanism of the infill walls, Pescara del Tronto

Figure 3.67 -From left to right: Out of plane failure of infills walls, Amatrice. On the right, out of plane of the coatings, Amatrice

3.2.9.3. Collapse of Exterior Panels in Double Leaf Walls

Figure 3.68- From left to right: external leaf overtuning and total overturning of the infill wall (on the right), Pescara del Tronto. On the right, external leaf overturning, total/partial overturning of the infill wall and coating's overturning . This building belongs to the seventies, time of fast and poor construction, explaining the level of damage. Other buildings in Norcia’s periphery present this kind of damage

Figure 3.69 -In this image is possible to see all the damage aforementioned:in plane failure (diagonal crack), out of plane failure (incipient external leaf's overturning, infill walls' overturning and coatings' overturning), Norcia

3.3. Row Buildings

Usually, italian historic centres are characterized by aggregations of masonry buildings, like the cases of Amatrice and the small villages. As a rule, only excepting irregular cases, masonry buildings are structurally linked to each other creating a block of buildings and, along the roads and streets, forming row buildings. This particularity of the historic centres complicates the interpretation of the buildings behaviour under seismic actions, because for a precise analysis of each structure it is also necessary to analyse its immediately adjacent structures, figure 3.73.

The interpretation of a building performance belonging to agglomerate is different from the interpretation of an isolated building, due to the interactions of its adjacent on the structural unit, figures 3.71 and 3.72. This interaction can be positive or negative: if it is positive the adjacent buildings act as buttresses or constraints but, on the contrary, if it is negative they introduce vertical loads or horizontal pushes figure 3.71. Consequently, these interactions can change the failure mechanism of the building by introducing new distinct actions and modifying the constraints arrangement.

Also, if one of the buildings is retrofitted or some modifications made it taller or heavier the risk for all row increases. Different structural systems induce hostile interactions due to their distinct stiffness of walls and/or floors. In the cases where structures are taller than their adjacent the collapse of the taller portion may occur due to hammering between the two units figure 3.72 a), detailed in section 3.1.4. Damage is often concentrated in the last building in the row, and can cause the collapse of the outermost building if the displacements are significant figure 3.72 b, (Giuffrè, 1993 and Carocci, 2001). For this reason, in the past buttresses or ties were introduced in this building, section 6.1.2.

Many times, the connections between adjacent buildings are poor or even absent because of the transformation phases, pre-existing discontinuities such as flues, the openings position (too close to the corners, extreme width and length of the spaces, or reduced distance between openings) and introduction of disconnected additions.

After an extreme event, such as an earthquake, the reconstruction of how the row building was formed is fundamental to evaluate the buildings vulnerability. In presence of cases of row buildings, the process of vulnerability evaluation must treat the structures as a whole. It can help understand the efficiency of the constraints between the walls and discover discontinuities between masonry portions.

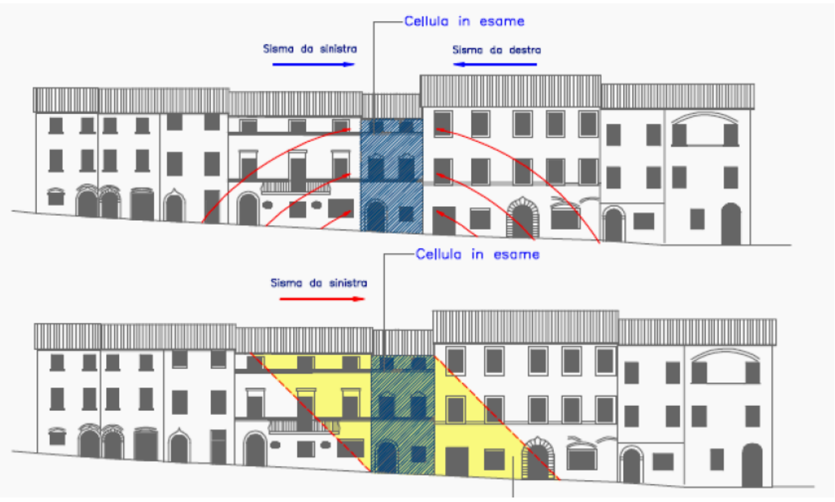


Figure 3.70 -Hammering from adjacent building, (Borri, 2004c)

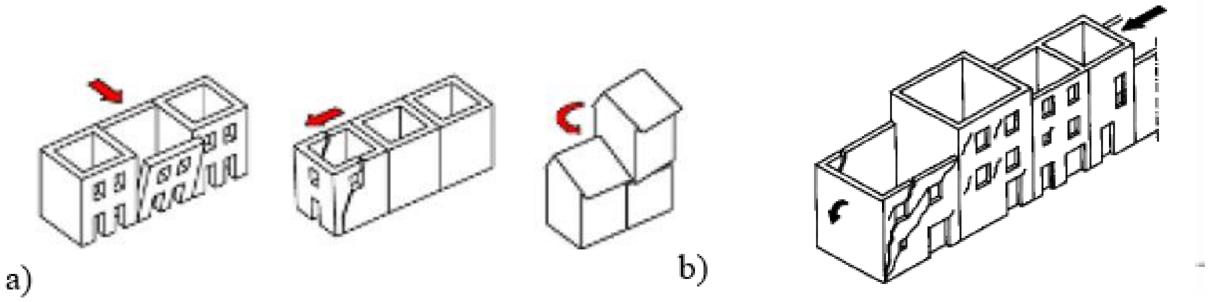


Figure 3.71 -From left to right: a) examples of damage mechanisms due to interactions between adjacent buidings. b) damage of the outermost building in a row

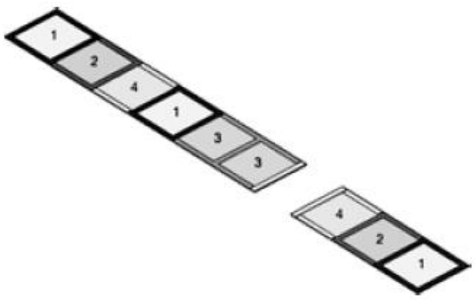


Figure 3.72 -Evolution of a row of buildings, (Carocci, 2004) 1 isolated building, 2 building built adjacent to the previous one; 3 contemporary buildings built adjacent to the previous ones; 4 building built between existing buildings, 5 demolished building

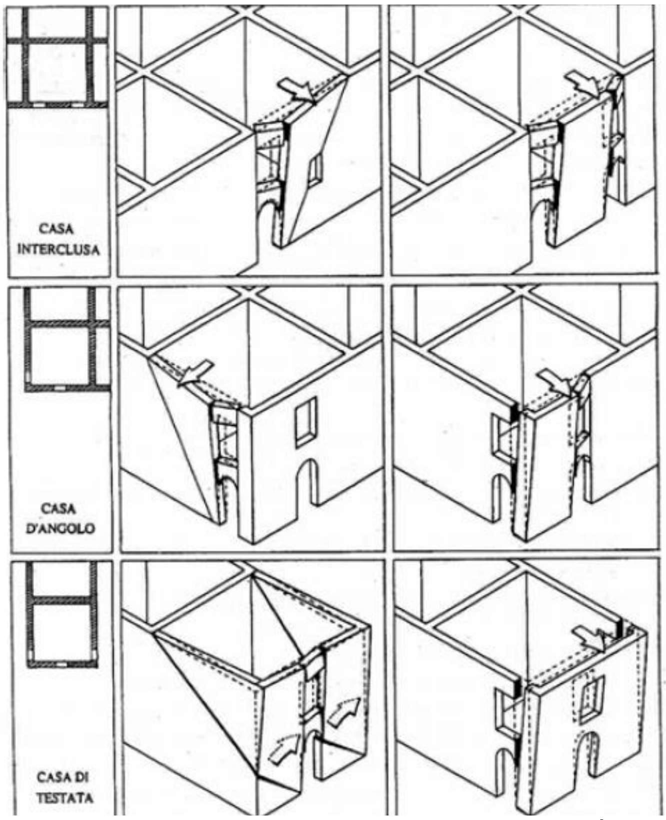


Figure 3.73- Damage mechanism for row buildings (Giuffrè 1993)

3.3.1. Row Buildings in Pescara del Tronto

Regarding the earthquake of 24th August, many buildings in a row were completely devastated in the several villages. The figure represents two examples of full ruin of entire rows along the road accessing Pescara del Tronto. It is not possible to evaluate the causes of collapse only based on photographic inspection due to the high level of destruction. What is possible to guess is that the buildings "interacted" during the earthquake and toppleed like "dominoes", even those which would have resisted the event were taken down by the others. Important to mention that, buildings constructed during diverse periods and with diverse materials behave differently.

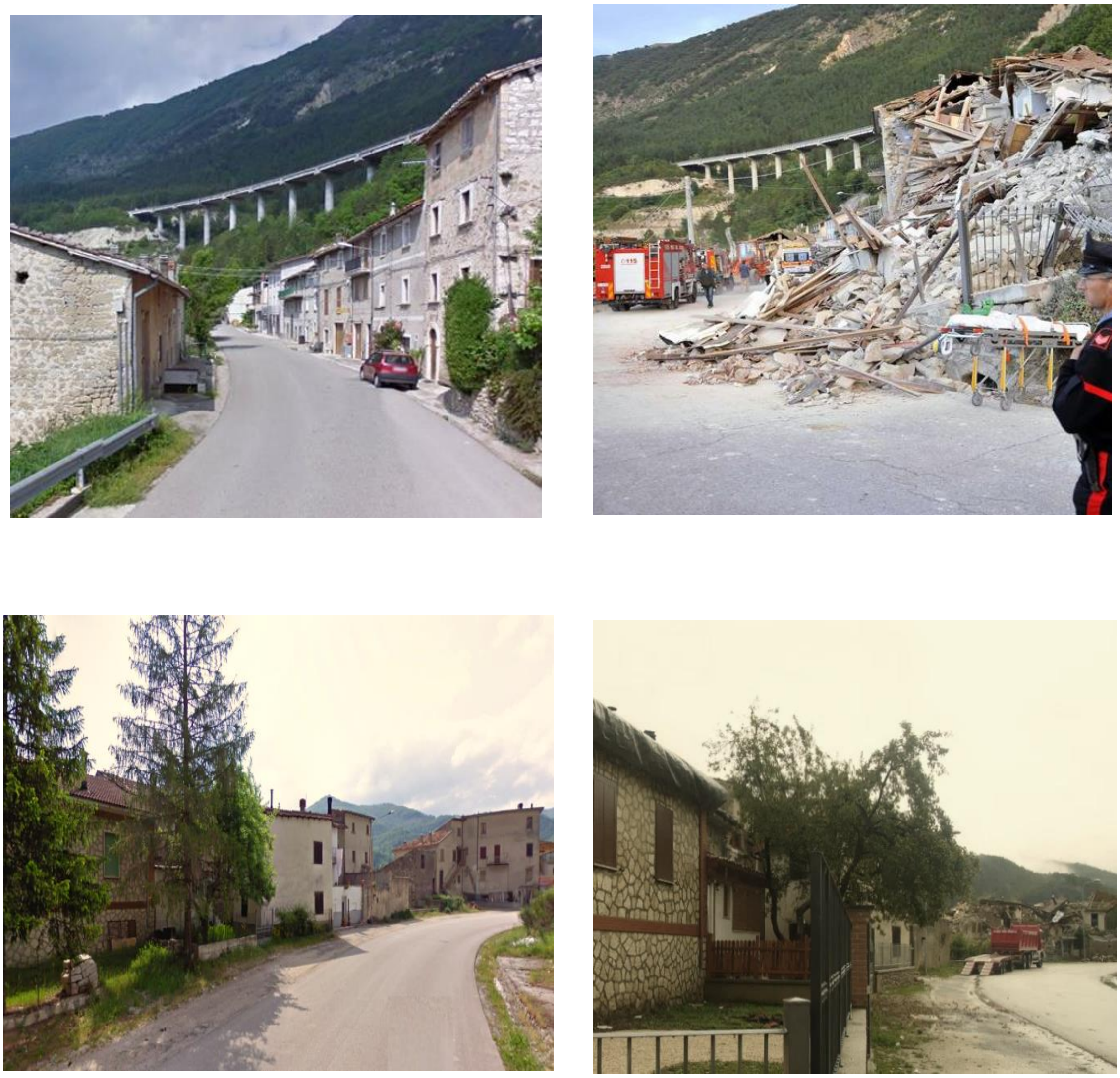


Figure 3.74 -Destruction of two entire rows of buildings in Pescara del Tronto

4
PERFORMANCE AND DAMAGE OF BUILDINGS STRUCTURES

4.1. INTRODUCTION

This chapter consists in assessing and categorizing the damage patterns of the damaged dwellings after the devastating earthquake, in the most affected areas of Amatrice, Pescara del Tronto and Arquata del Tronto, based on the photographic data. All three villages were located on steep mountain edges where slope instabilities, like landslides and rock failures section 2.4, and site amplification can provoke significant damage, as it has been well documented during past events in Italy.

Pescara del Tronto was the most damaged village within the three here documented, remaining practically nothing from the old hamlet, and for this reason, a more detailed report is done. Amatrice was severely damaged in the SE zone in the historic centre with few buildings remaining standing, including the reinforced concrete building aforementioned. Arquata del Tronto suffered damage mainly on top of the ridge, including some full collapses but, many buildings behaved well, especially away from the edge.

Most of the built of the three villages that collapsed were unreinforced masonry structures provided with wooden floors. Reinforced concrete buildings also suffered severe damage. A detailed evaluation of each structure is reported and classified, following the scheme provided by the Department of Civil Protection (DCP) for damage inventory purposes in post-earthquake situations figure 4.1. The table presents the scheme for post-earthquake reconnaissance which includes five levels of damage from D0 (no damage) to D5 (total collapse). This classification is based on the geometric characteristics of buildings such as, number of floors, area or height, type of building, masonry, reinforced concrete frame, reinforced concrete walls or steel structure and soil conditions/foundations. The damage quantification is done according to its extension on the structure and on which structural elements are involved (vertical elements, horizontal elements, roof and walls).

Damage Level	Description	Marker Color
D0	No damage	
D1	Cracking of non-structural elements, such as dry walls, brick or stucco external cladding	
D2	Major damage to the non-structural elements, such as collapse of a whole masonry infill wall; minor damage to load bearing elements	
D3	Significant damage to load-bearing elements, but no collapse	
D4	Partial structural collapse (individual floor or portion of building)	
D5	Full collapse	

Figure 4.1-Definition of damage categories (adapted from Bray and Stewart, 2000)

Istat compiled the following table where it classifies the state of conservation of the buildings of Amatrice and Arquata del Tronto (Pescara del Tronto belongs to Arquata del Tronto), based on the data collected during the census in 2011. Most of the buildings was classified as good/excellent which was not confirmed after the earthquake on the 24th August.

Table 2-Residential buildings gathered by its time of construction and state of conservation, year 2011 percentage values

Villages with significant damage	Before 1971		1971-2011		Total
	Good/Excellent	Bad/Terrible	Good/Excellent	Bad/Terrible	
Amatrice	59,5	16,4	23,3	0,8	100
Arquata del Tronto	73,8	13,5	11,6	1,0	100

Each sub-chapter of each village includes a short description of the villages, geological conditions cited from the GEER report, map of the towns with the locations of damage representative buildings and its classification according to the DCP scheme, pictures from the representative buildings and a following table that describes the damage visualized. The photographic report exhibited illustrates the damage patterns found, which were not very different from village to village due to the similarity of their built. In Amatrice this report is divided in two: the red zone and the suburbs photographic report due to the distinct typology of the structures: in the red zone prevailed the masonry building whereas in the suburbs the reinforced concrete structures were quantitatively well represented. A damage zonation map was also elaborated for the villages of Arquata del Tronto and Pescara del Tronto.

Figure 4.2 - Panoramic picture from Pescara del Tronto

Figure 4.3 -Panoramic picture from Amatrice

Figure 4.4 -Panoramic picture from Arquata del Tronto

4.1.1. AMATRICE

Amatrice is a small town, with 2650 habitants, situated in a basin, part of Rieti, in northern Lazio, Central Italy. The epicentre occurred 15 km away from the town. Many *frazioni* were included in its territory which were also devastated like Cascello, Voceto, Mosicchio, Casale, Saletta, Sommati, Villa San Lorenzo Flaviano, Callecreta, Montegallo and Montereale.

The storey of Amatrice begins in prehistoric times denounced by archaeological findings, continues throughout the Roman Empire and, from the medieval and early modern periods, there were some exemplars, mainly churches and sanctuaries, dating from the 13th century until the late 15thcentury. Also, some buildings of artistic and cultural interest date from the 16th until 18th century. Nowadays, Amatrice's economy was relying on tourism and agriculture.

The centre of Amatrice is situated above the sea level 900 to 1000 m and above the confluence of both Tronto and Castellano rivers. Regarding the geological conditions, "it lies on terraced alluvial soil of lacustrine-fluvial origin, which overlap the local substrate of Laga Flysch" (source GEER).

Amatrice town was only classified as seismic area after 1915.

4.1.1.1. Photographic Report

The following photographic report is divided into two: red zone (city centre) report and periphery report. These pictures from Amatrice were collected from a GEER report and date from 9 to 13 September 2016. The field survey was led, in first place, in the suburbs of Amatrice due to the inaccessibility of the historic centre. Once in the red zone, the team only had access to precise streets such as, the main street Corso Umberto I, some perpendicular streets to the main street and other roads around the inner perimeter of the area.

The so-called red zone is the historic centre of Amatrice which experienced mainly full collapse of masonry buildings, being the average classification level D5. In the suburbs, the level of destruction was not so high mostly because of the good behaviour of some reinforced concrete structures although, there were reported some masonry buildings collapse.

4.1.1.1.1 Periphery Photographic Report

As usual, the suburbs are comprised of a higher number of reinforced concrete structures than historic centres, explaining the diversity of damage found in this zone. Regarding site amplification "there were no clear patterns of damage that would suggest site amplification effects" (source GEER).

Generally, masonry buildings collapsed totally or partially (image P1 P2 and P3) and reinforced concrete buildings only suffered non-structural damage like in plane shear failure of infills and crisis of the beam-column joint (image P3b, P4 and P6).

The police station building, P5, is a masonry building that did not collapsed but suffered many in plane damage (shear cracking in both pier and spandrel panel). With a closer look is possible to visualize the steel ties close to connection between the roof and the walls hence, seems reasonable to state that this was a case of positive retrofitting and the building "behaved like a box". The images P3 belong to the same building, the school Romolo Capranica that, despite previous seismic interventions (only a five years ago) did not survived the August earthquake. This building was composed by both masonry and reinforced concrete corps and, as it is visible in the images, the masonry part collapse while the reinforced concrete part remained standing with non-structural damage. The hospital F. Grifoni, P8, was also composed by portions of masonry and reinforced concrete and, in this case, the response of the buildings was better than in other cases, although considerable damage to both structural and non-structural components were reported. The masonry part exhibited extensively in plane shear failure in both pier and spandrel panels and, in the worst case, below presented, one wall presents signs of an incipient out of plane mechanism. The reinforced concrete part exhibits diagonal cracking and separation between infills and structural frame accompanied by coatings detachment.

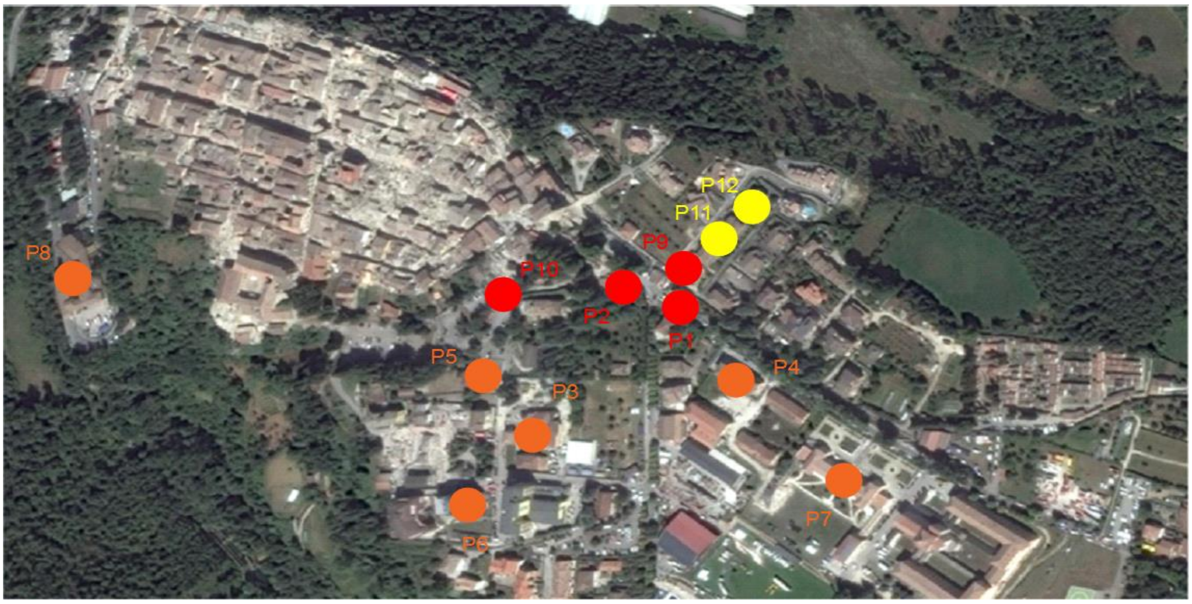


Figure 4.5 -Locations of the reported structures and their level of damage: yellow-D2, orange-D3 and red D4 and D5

Table 3-Images, respective damage level attributed and damage description, Amatrice

Image	Damage Level	Damage description
P1	D4	Collapse of both floor and façade in a masonry building
P2	D5	Full collapse of a masonry building with RC roof
P3	D4	Partial collapse of the masonry part of the school Romolo Capranica
P4	D2	Satisfactory behaviour of the RC part in school, only suffering non-structural damage
P5	D3	Severe in plane shear failure, X-cracking, in the police station
P6	D2	Non-structural damage in a RC building, out of plane of the infills
P7	D2	Non-structural damage, detachment of the plaster in the masonry building
P8	D3	Hospital, severe non-structural damage in the RC part (8a) and extensive in plane shear failure in the masonry part with incipient out of plane failure (8b)
P9	D5	Hammering of the RC roof in the masonry walls
P10	D5	Full collapse of a masonry building
P11	D1	Minor non-structural damage
P12	D2	Non-structural damage, separation of the infills from the RC frame

P1

P2

P3a

P3b

P4

P5

P6

P7

P8a

P8b

P9

P10

P11

P12

Figure 4.6 -Images took from suburbs of Amatrice, representative of the area in terms of damage

4.1.1.1.2 Red Zone Photographic Report

The red zone was mainly damaged in the SE part (figure 4.7) where many masonry buildings fully collapsed, being some of them masonry buildings in the main street (pictures P6 and P8). On the contrary, some recently seismic intervened masonry buildings behaved well (picture P3 and P4) surviving the earthquake without damage or with minor damage. The Hotel Roma (picture P1), the town's principal hotel suffered a brittle failure, soft storey mechanism and, from the three-floor building, was only left one, taking the life of six tourists. As aforementioned, in this area the only building that did not collapsed was the reinforced concrete structure. (picture P7), although it suffered major damage. Near the Hotel Roma, was located a steel frame (picture P2) that also had a positive behaviour against the seismic action, only suffereing non-structural damage and incipient buckling of the columns. The damage map (figure 4.8) was elaborated by the GEER team.

Besides the red zone located in the historic centre, another red zone was declared a few metres from the main destruction (figure 4.10).

Figure 4.7 -Before the earthquake on top and after the earthquake on the bottom. The level of destruction is easily visible especially in the east zone

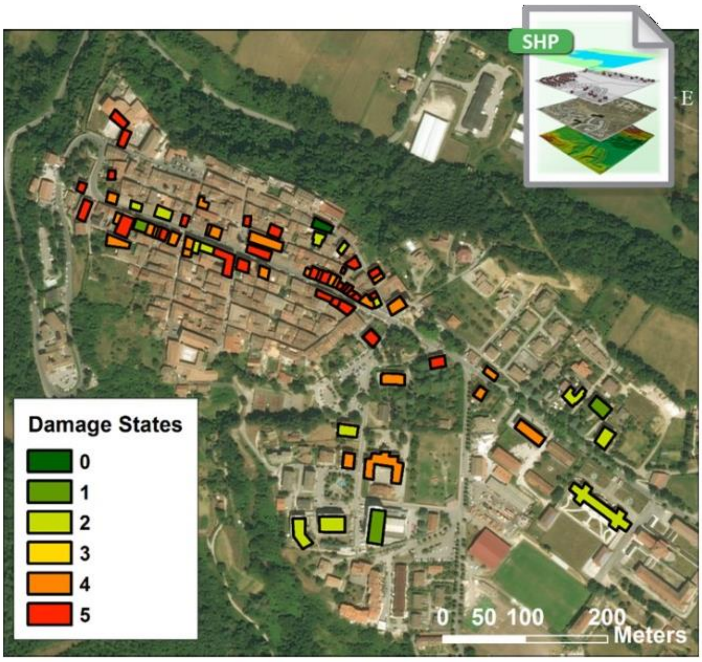


Figure 4.8--Overview of the damges in Amatrice seen by aerial image
(source GEER)

Table 4- Images, respective damage level attributed and damage description, Amatrice

Image	Damage level	Damage descripition
P1	D5	Soft storey in Hotel Roma
P2	D2	Severe non- sructural damage in a steel frame and incipient buckling of the columns
P3	D1	No visible damage
P4	D1	No visible damage
P5	D5	Partial collapse of a masonry building
P6	D5	Full collapse of a building with RC roof
P7	D2	Severe non-strctural damage to a RC building
P8	D5	Full collapse of masonry buildings
P9	D4	Full collapse of masonry buildings
P10	D5	Full collapse of masonry buildings

P1

P2

P3

P4

P5

P6

Figure 4.9 -Images took from Amatrice's historic centre, representative of the area in terms of damage

Figure 4.10-The two red zones in Amatrice

4.1.2. ARQUATA DEL TRONTO

Arquata del Tronto is a municipality in the province of Ascoli Piceno, in the Italian region of Marche and located 11 km from the epicentre of the 24th event. It is believed that 57 people died in the night of the earthquake. Arquata first mention in history dates from the middle ages (6th century) when a stronghold existed. This village englobes a significant number of hamlets such as Pescara del Tronto, Borgo, Tufo and Vezzano. The village Arquata is situated in an extended WNW-ESE ridge, 730 meters above the sea level. The ridge is constituted by "laga flysch and eluvial/colluvial gravelly soils and fluvial gravels and sands cover the flyschoid bedrock NE and SE of the ridge" (source GEER). Phenomena of slopes instability occur in this area involving the type of rock described.

Regarding Arquata seismic history, the village was hit by the 1703 Valnerina earthquake and the intensities measured were IX MCS and, during the 1916 Monti Sibilini event intensities reported were VII MCS. The first earthquake had a magnitude of M 6.9 and the second M 4.8.

Arquata del Tronto municipality was only classified as seismic area in 1983.

4.1.2.1. Photographic Report

This sub-chapter, in the first part presents a map of the village with the locations of the damage representative buildings (figure 4.11) and a table with the classification of buildings according to its level of damage after the photographic report.

After the main shock, the village was significantly injured. Arquata del Tronto, founded on the top of the ridge, is a village comprised of masonry buildings with number of floors ranging between two or three and, few of which, exhibited retrofit measures. However, in cases of full collapse is difficult to observe the existence or not of retrofit instruments.

In case of picture P5, the structure exhibited a heavy concrete roof that led to its collapse. The adjacent building (picture P3) behaved completely different only suffering in plane shear failure with detachment of plaster in the pier panels. Cases of total and partial collapse (pictures P1, P4, P7 and P13) occurred but also many masonry buildings behaved well during the event (pictures P2, P3, P11 and P12). Cases of failure of retaining walls were also reported (picture P6) along the road accessing Arquata.

As it is visible in the images, the level of damage seems to diminish away from the ridge. Another village a few meters from Arquata del Tronto, called Borgo, suffered minor damage, mostly in plane failure of the non-structural elements and is possible to find buildings retrofitted with iron bars. To investigate the possibility of site amplification due to the different damage found in the two hamlets, the GEER team decided to carry out four noise measurements. The conclusion was that the significant damage in the ridge may be partially associated with topographic amplification effects but, also the significant vulnerability of Arquata's unreinforced masonry buildings had an important role. These two characteristics combined (buildings vulnerability and site amplification) explain the differences found in both locations. In the 1703 earthquake event this phenomenon also occurred: Arquata del Tronto suffered significant damage IX MCS, and in the Borgo hamlet the reported intensities were VII-VIII MCS.

A possible division of the village according to its level of damage is reported in the figure 4.13. The most injured area, on top of the ridge, is classified as D4-D5 as some buildings fully or partially collapse. However, there were also good seismic exemplars located in the same area classified as D2. The level of damage diminished going in the east direction from D4-D5 to D2-D3, where the buildings P8 are located. Moving away from the edge in both east (pictures P11 and P12) and north direction (Borgo), the level of damage decreases from D3 to D3-D2 and D1-D2.

After the seismic events in October the area of the ridge was destroyed, D5 level, section 5.2.3.

Figure 4.11- Locations of the reported structures and their level of damage: yellow-D2, and red D4 and D5

Table 5-Images, respective damage level attributed and damage description, Arquata del Tronto

Image	Damage level	Damage description
P1	D4	Partial collapse of a masonry building
P2	D2	Non-structural damage including detachment of the plaster and incipient out of plane-vertical crack along the intersection
P3	D2	Severe non-structural damage in pier panels to a masonry building
P4	D5	Full collapse of masonry buildings
P5	D5	Full collapse of a masonry building with RC roof
P6		Collapse of a retaining wall
P7	D4	Out of plane mechanism of the front façade involving two floors
P8	D3	Panoramic view, masonry buildings with significant structural damage
P9	D5	Panoramic view, masonry buildings with significant structural damage
P10	D3	Severe cracks along the intersections that may trigger out of plane mechanisms
P11	D1	Minor non-structural damage
P12	D1	Minor non-structural damage
P13	D4	Partial collapse of a masonry building

P1

P2

P3

P4

P5

P6

P7

P8

P9

P10

P11

P12

P13

Figure 4.12-Images took from Arquata del Tronto, representative of the area in terms of damage

Figure 4.13-Damage zonation, Arquata del Tronto and Borgo

4.1.3. Pescara del Tronto

Pescara del Tronto is a municipality of 122 inhabitants of Arquata del Tronto and located 6 km northeast from the epicentre of the earthquake. In this village 48 people died in the night of the event. Only four families lived permanently in Pescara but, in the summer the number could rise until 300 people.

Pescara is situated 743 above the sea level and geologically "the site is complex due to convergence of the two mountain ridges: a calcareous ridge of the Sibillini Mountains and a turbiditic ridge of the Laga Mountains, oriented respectively NNE-SSW and NW-SE" (source GEER). Both Arquata del Tronto and Pescara del Tronto are situated between two protected natural areas, the National Park of Sibilini mountains to the north and the Gran Sasso National Park and Laga mountains to the south.

Regarding its seismic history, the few information available documented damage from two strong earthquakes, one is the 1703 earthquake M 6.9, with intensities attaining the level IX MCS. More recently the 1941 earthquake M 5.0 reported VII MCS damage, (Rovida et al.,2016).

4.1.3.1 Photographic Report

The majority of the built in Pescara was unreinforced masonry structures with two/three floors and only few of which were retrofitted. A survey conducted by the INGV team after the earthquake, classified most of the built, about 60%, as mixed, which consisted mainly in masonry built on the first floor and reinforced concrete in the rest of the structure. Only four buildings in Pescara del Tronto were reinforced concrete buildings, table 7. This survey also documented the poor quality of masonry that was a constant in the area affected by the earthquake: "irregular limestone block walls were frequently found, sometimes mixed with brick elements, mortar quality was also checked and found to be very poor. Mortar generally appears to be made up of sand and hydraulic lime". This type of mortar provides a very weak connection between stones. Many buildings also suffered addition of storeys.

Regarding site effects in Pescara del Tronto, the report of GEER team stated that the high level of destruction of the hamlet was not due to topographic effects, like Arquata del Tronto, because the cliff was less significant and the covering soils were small. However, a further deep analysis is needed also because the noise measurements were not located in the most affected area due to its inaccessibility. The occurrence of many landslides also influences a lot the level of devastation in the small hamlet.

The locations of the representative buildings of damage are shown in the map (figure 4.14) and in the table the classification according its level of damage. In this village, many damaged areas were not accessible but the main destruction was seen from the road accessing Pescara.

Picture P11 represents the only retrofitted building found in the village of Pescara and it behaved well under the seismic action. In picture P8, it is well visible the different behaviours of two adjacent buildings: one is the building P11 that only experienced a detachment of material in the corner zone, while its adjacent completely collapsed. Retrofitting measures in the first structure may have saved lives. The most damaged area was the southern part of Pescara del Tronto, with full collapse or partial collapse of the buildings totality (picture P9 and P13). Along the road, the failures seemed slightly lower (picture P3 and P7) although there were also cases of partial collapse (picture P5 and P6). Away from the main agglomerate it can be detected isolated reinforced concrete buildings along the road, which suffered major damage (picture P1): besides the evident out of plane of the infills, the building suffered severe damage in the beam-column joint in especially one column.

Damage patterns were documented in figure 4.16 in a 3D model of Pescara elaborated by the GEER team. This model clearly shows the various landslides occurred, and in evidence is the main landslide due to a retaining wall failure. It is also visible the full destruction in the southern area, while the buildings along the road behaved better during the seismic action. As stated previously, the severe damage in Pescara del Tronto was accompanied by large landslides deforming the ground permanently, which caused even more destruction to the village.

In order to make a damage zonation map figure 4.17, the damage patterns were analysed. The first possible conclusion is that the minimum level attributed is D3, excepting two buildings categorized as D2 (figure P3 and P7), demonstrating the high level of destruction that Pescara del Tronto was submitted during the seismic event. The northern area, that includes the road accessing the village, was divided into two different failure levels, D3 and D4. The first level D3 includes buildings with slightly minor damage P1, P2 and P4, and buildings with damage only in the non-structural elements, P3. The level D3/D4 includes the buildings which collapsed partially, P5 and P12. To the southern area was attributed both D4 and D5 as all the buildings partially or fully collapsed, P8, P9, P10 and P13.

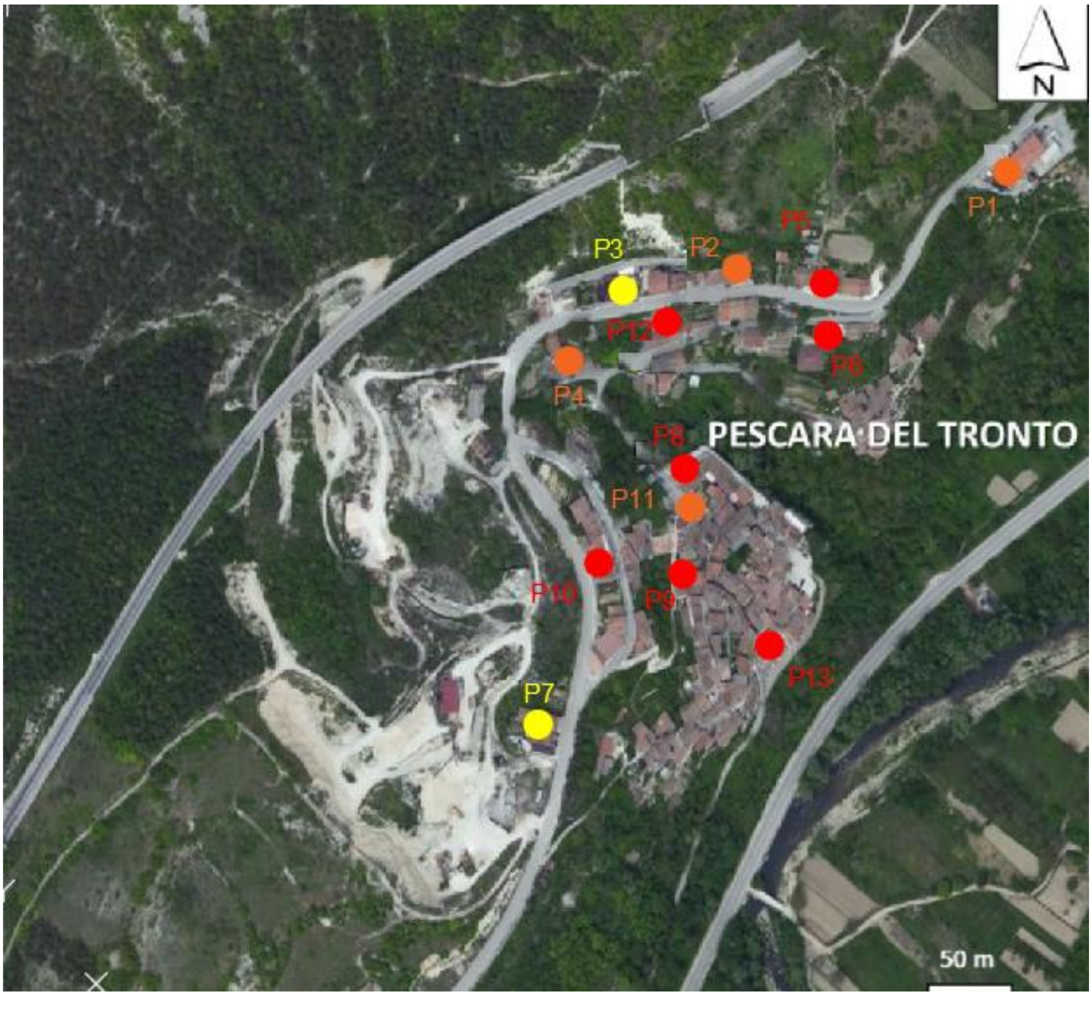


Figure 4.14 -Locations of the reported structures and their level of damage: yellow-D2, orange-D3 and red D4 and D5

Table 6- Images, respective damage level attributed and damage description, Pescara del Tronto

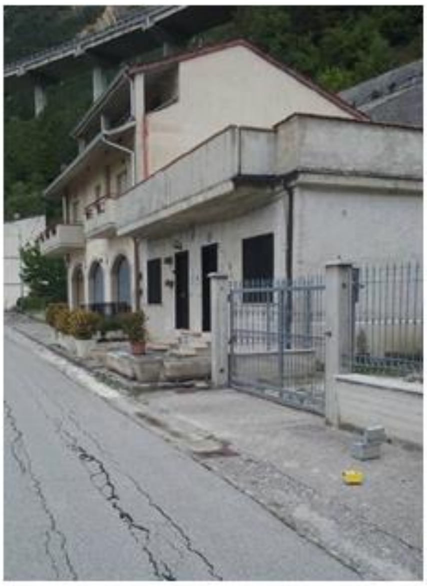
Image	Damage level	Damage description
P1	D3	Severe non-structural damage to a RC building, out of plane failure of infills and damage in the beam-column joints
P2	D3	Out of plane mechanism in the connection between roof and walls in a masonry building
P3	D2	Damage to non-structural elements
P4	D3	Out of plane and in plane failures and roof collapse
P5	D4	Partial collapse of a masonry building
P6	D5	Full collapse of a masonry building with RC roof
P7	D2	Sever non-structural damage in a masonry building, out of plane of infills
P8	D5	Full collapse of a masonry building with RC roof
P9	D4/D5	Panoramic view of fully collapsed masonry buildings
P10	D4/D5	Full collapse of a masonry building
P11	D3	Corner detachment below the tie rods
P12	D4	Partial collapse of a brick masonry building
P13	D4/D5	Panoramic view of fully collapsed masonry buildings
P14		Effects of landslides

Table 7 Data extracted from the 2011 Italian population and housing census (ISTAT, 2011), (source INGV)

Typology of buildings	Masonry	37
	Reinforced concrete	4
	Mixed	69
Age of buildings	<1919	88
	1919-1960	14
	1961-1980	6
	>1980	2
Number of floors	≤ 2	47
	3	59
	≥4	4
State of preservation	Excellent	14
	Good	65
	Mediocre	21
	Very bad	10

P1

P2



P3

P4

P5

P6

P7

P8

P9

P10

P11

P12

P13

P14

Figure 4.15- Images took from Pescara del Tronto, representative of the area in terms of damage

Figure 4.16-3D model from drone flights on the village of Pescara del Tronto

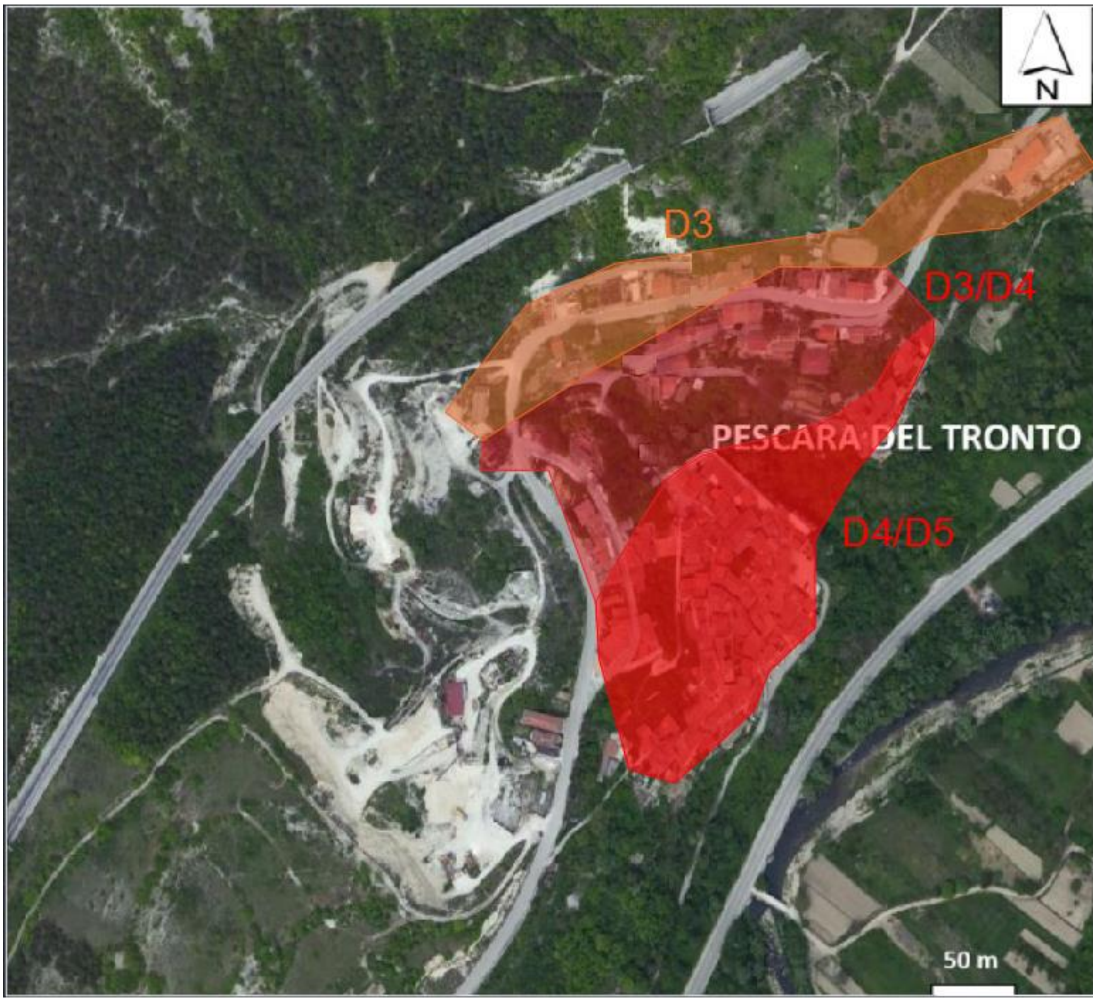


Figure 4.17 -Damage zonation, Pescara del Tronto

4.2. DAMAGE LEVEL COMPARISON BETWEEN AUGUST 24 AND OCTOBER 26 AND 30

The following table compiled by the GEER team, makes a comparison between the damage levels after the August 24 and after the October 26 and 30. As expected the damage level attained the higher level D5 in all villages in October due to the previous weakening of the structures in August. Even though some structures survived the first earthquake, during the October events they failed.

In Arquata del Tronto, that was the hamlet presenting many buildings with minor failures was completely devastated in October. Regarding Amatrice, it is only considered in the table the red zone.

Table 8-Comparison of roughly mean damage levels after the 24 August and the October events (source GEER)

Village	Damage level 24 August	Damage level 26 and 30 October	Note
Amatrice (red zone)	D4-D5	D5	Controlled demolitions after September
Pescara del Tronto	D4-D5	D5	
Arquata del Tronto	D3-D4	D5	

5
CAUSES OF FAILURE MECHANISMS

After the previous classification of the buildings in five different categories depending on their level of damage, now this chapter aims at exhibiting detailed photographic data and group the damage patterns according to its cause of failure. This data collection was conducted in the three villages before analysed Amatrice, Pescara del Tronto and Arquata del Tronto and in this chapter Accumoli and Norcia are added. Besides the failure causes reconnaissance, a comparison of the villages before and after the seismic event is equally important to understand the large impact of this earthquake. With the help of Google Maps 2011, images from the pre-earthquake were obtained from the villages Amatrice, Pescara del Tronto and Arquata del Tronto. As many damaged buildings were not achievable, their registration was only possible through the consultation of previous photographic reports of the 24th August.

5.1. Most Frequent Deficiencies in Unreinforced Masonry Buildings

5.1.1. Poor Quality of Materials and/or Heterogeneity of Materials

The quality of the masonry determines the capacity of the building to bear vertical and horizontal forces from seismic actions. As discussed in section 3.1.1.3, very poor quality masonry is characterized by small size rubble mixed with bigger elements, all randomly placed, without headers and weak mortar. The tiny dimensions of rubble allied to the poor quality of mortar prevent a good connection along the thickness of the wall. Figure 5.1 right shows one example of a poor mortar, in this case soil mortar which does not allow a good performance under the seismic action.

During the seismic action, these irregularly placed stones tend to move out from the wall causing localized damage or even its collapse, in extreme cases, figure 5.2 a) and b). In these figures, it is visible the presence of stone elements of minimum size disconnected from the mortar, which reveals two deficiencies: the low quality of the mortar and the absence of so-called headers, meaning that the leaves are not transversely connected. Figure a), on top of these features, also exhibits round shape river stones whose link with mortar is much weaker comparing to squared shape stones. Figure b) the bad state of maintenance also compromises the rest of the building.

Regarding the figure d), the building was found to have been subjected to additional construction by adding one or two storeys, because the ground floor is made up with limestone and the upper storeys are hollow clay bricks called *occhioni*. These bricks, which are improper for structural purposes, are also placed in horizontal courses diminishing even more their resisting capacity. The building in figure c) is also made up with the same inadequate bricks in horizontal courses which led to its partial collapse.

Besides the poor quality of the materials, many times is also detected in some buildings (figure e) and f)) the combination of different materials, like stone and brick. Because of the differences in size and shape of the units, the link between orthogonal walls is inadequate. Figure f) shows a building in which, one wall is constructed with brick masonry and the other with stone masonry. The use of mixed structural units and systems results in variable wall strength and stiffness in different parts of a building, causing torsional effects once damage begins to accumulate in the building. It is acceptable to mix materials if each material is used for each storey and stronger materials are used for the ground floor walls construction.

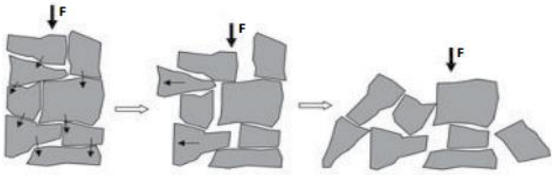


Figure 5.1 -Detail of wall failure caused by irregular stones (Bothara and Hiçyılmaz). On the right, a building collapse due to its poor quality masonry, which included soil mortar

a)


b)


c)


d)


e)


f)


Figure 5.2-Visible poor quality of the masonry in a), b), c) and d). Pictures e) and f) besides the poor quality of the masonry presents heterogeneities in the masonry

5.1.1.1. Lack of Connections Between Leaves

The following figure 5.4 represents cases of external leaf overturning in a double leaf masonry wall due to the poor quality of masonry and the absence of the so-called headers.

Delamination takes place when vertical wall layers bulge and collapse due to earthquake ground motion. The thick walls are built in multi-leaf configuration with poor construction techniques, without headers or interlocking between the leaves. This is the typology of walls generally vulnerable to delamination due to its decreased integrity. In addition, many times, the exterior leaves can be made with properly dimensioned brick units, whereas the middle layer is filled-up with rubble bricks, as already mentioned in section 3.1.1.3.

A detailed experimental and analytical research study on the delamination of stone masonry walls was performed by Meyer et al. (2007) and, according to the study, delamination is triggered by high frequency vibrations which cause inter-stone vibrations. This results in a reduction of frictional forces which maintain the stones together. Delamination is usually initiated in the upper portion of the wall, and the damaged wall looks like it was "peeled off". After the 2002 Molise earthquake, Decanini et al. 2004 reported that "spreading (delamination) damage in stone masonry walls begins at the top of the building, where the lack of overburden weight allows the masonry to vibrate apart. The stability of the wall can be most at risk when the masonry units vary in size and are laid with a minimum of horizontal bedding". The presence of headers, one again, plays a fundamental role in preventing this type of failure.

Another type of disconnection between leaves happened in one church in Norcia where the type of masonry was a three-leaf wall, with limestone inside and regular stone on the outside, section 3.1.1.3. In the figure 5.5 it seems to have occurred saline efflorescence which weakened the masonry wall and facilitated its delamination.

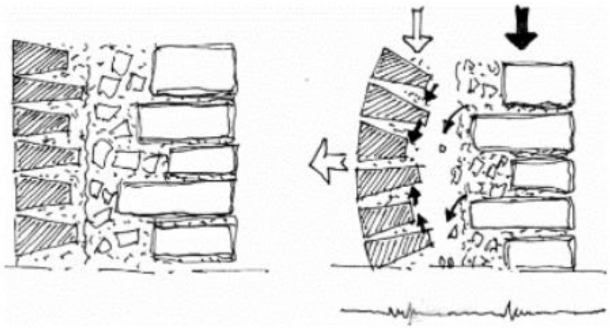


Figure 5.3-Delamination of multi-wythe wall (Nienhuys, 2003)

Figure 5.4 -External leaf overturning in Pescara del Tronto and Arquata del Tronto

Figure 5.5-Saline efflorescence
in one church in Norcia

5.1.2. Absence of Diaphragms and/or Lack of Connections Between Horizontal and Vertical Elements

In ancient constructions, the absence of diaphragms was a common practise, inserted in a context of poor construction techniques. Only from photos analysis is not possible to understand if there is, or there is not, the presence of diaphragms, it is necessary a local visit. However, it seems reasonable to take into account that most of the masonry built had this deficiency, and reports of GEER and RELUIS mention this topic. The seismic code demands at least 5 cm of thickness of the reinforced concrete slab and in most cases the slab was thinner or absent.

In order to permit a reasonable behaviour, is necessary to improve the connections between horizontal (floors and roof) and vertical elements (masonry walls). Connections between structural elements are a key aspect for effective seismic resistant construction. Ensuring the 'box behaviour' of a building, where the horizontal forces are absorbed by walls in the same plane, is the most effective measure against earthquakes, section 3.1. When the walls are not connected at the intersections, each wall is expected to vibrate on its own when subjected to earthquake ground motion, figure 5.6. In this situation, the walls perpendicular to the direction of the shaking are going to experience out of plane vibrations and possibly collapse when connection between roof and walls is not adequate. Walls parallel to the direction of the shaking, shear walls, are also susceptible to damage. When the walls are well connected, by a rigid roof, and/or a horizontal ring beam at the slabs, the building acts as a monolithic box which is the satisfactory seismic performance, as aforementioned, in the section 3.1.

The next sub-chapters are divided in collapse of façades and collapse of floors, which means that lack of connections between walls and floors/roofs only activates out of plane mechanisms.

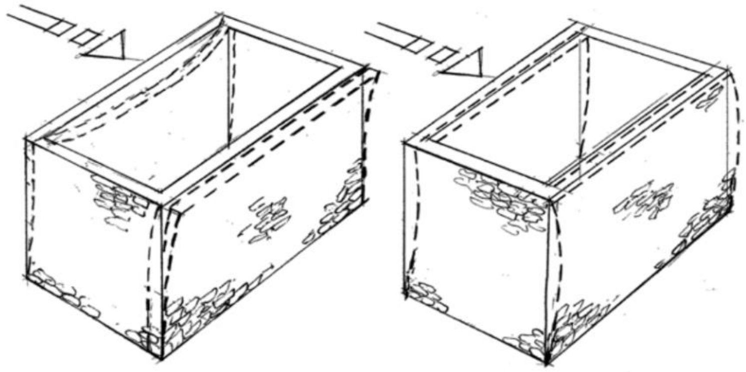


Figure 5.6 -Masonry building during earthquake shaking: a) loosely connected walls without slab at the roof level, and b) a building with well connected

5.1.2.1. Partial or Full Collapse of Façades

In the figure 5.7 is well visible the absent connection between the floors and the walls (pointed by the red arrows) causing the full overturning of the façades. In addition, the excessive thickness of masonry walls, result in significant inertial forces that facilitates the collapse of façades by simple or partial overturning. The bad quality of masonry described in section 3.1.1.3 is visible in all pictures. In figure a) the only good connection is the one between roof and walls.

a) b) c) d)

Figure 5.7-Overturning of the front façades due to lack of connections between walls and slabs

5.1.2.2. Collapse of Floors

Floors collapse due to failure of connections between the floors and the walls (figure 5.8), causing the overturning of the perpendicular wall and loss of support of the floors beams. Once again, the poor masonry in the pictures is a constant. In most cases, bonds between the floors beams and the walls are not correctly done and under the seismic action these defects come to the surface as slippage between the beams and the wall, figure 5.9. In figure d) the floors are still supported by the walls opposite to the ones which collapsed.

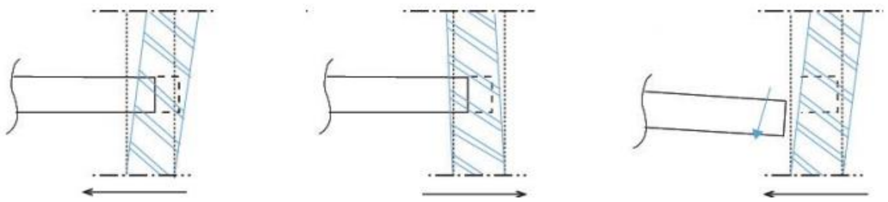


Figure 5.8-Behaviour of a possible connection during earthquake excitation (Nienhuys, 2003)

Figure 5.9-Collapse of floors due to slippage of the beams

5.1.3. Heavy Reinforce Concrete Roofs

A frequent form of upgrade found in the affected areas was the replacement of former wooden roofs by new roofs of reinforced concrete beams. Wooden roofs aimed at performing two main tasks: first, to connect and to couple the surrounding walls; second, as a flexible horizontal element. For a long time, reinforced concrete roofs, aimed at making the diaphragm more rigid and stronger but, however, this alteration only caused many collapses in earthquakes, as instead of damping the vibrations and dissipating the energy, the heavy roofs increased the horizontal forces and hammered the masonry walls. As the concrete and masonry have different vibration frequencies, they vibrate independently and the consequences are significant. This process was often executed without upgrading the masonry load bearing walls. Sometimes the load bearing walls were even removed to open up spaces which resulted in beams situated where shear walls once existed. During the post-earthquake survey it was found that rarely these alterations were according to regulations and seismic design criteria. The increase of weight and stiffness in the roof plan allied, in many cases, to the defective connection between roof and columns, causes the walls explosion. Moreover, the rise of the stiffness on top of the roof can obstruct the natural vibration mode of the building, encouraging local high stresses in this area. A major problem of this technique is that the replacement of old floors for new floors gives the wrong idea of security.

This being said, in masonry buildings, the ideal condition is to have light, rigid and resistant roofs and well connected to the walls: roofs able to transmit low inertia forces (lightness) and to redistribute the seismic forces among the walls parallel to the actions, being at the same time an efficient constraint for the out of plane wall overturning. The incompatibility of materials and systems is a good indicator of a building vulnerability.

The following pictures are all examples of this type of failure. In some pictures, the walls totally collapsed while in other cases the walls remained standing, figure 5.11. The example of the figure f), illustrates the case where the concrete roof hammered the masonry walls leading to the collapse of the upper floor.

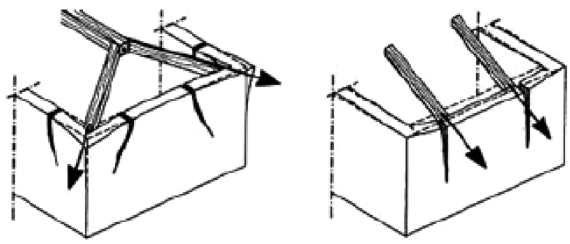


Figure 5.10 - Hammering of the roof on the load bearing walls, (Doglioni, 1999)

Figure 5.11 -Collapses due to heavy reinforced concrete roofs

5.1.4. Lack of Confinement or Insufficient Confinement in the Buildings Corners and Presence of Openings

Corner damage is common in masonry buildings. This type of mechanism generally occurs at wall-to-wall and wall-to-roof connections when subjected to out of plane displacements. During an earthquake, the stress concentrations increase at the intersection of the walls leading to vertical and/or diagonal cracks in the corners of the building. If the confinement between the two walls is not properly done, the intensity of the cracks rises and these cracks spread along the wall (figure 5.12 a), b), c), d) and e)). In the case of figure b) the diagonal crack caused an incipient detachment of the corner and in figure a) the diagonal crack occurred between the corner and the opening causing the overturning of the external leaf. Here occurred the interaction between in plane and out of plane failures. In figure d) the quoins introduced in the corners were not able to prevent the vertical cracks along the intersection between orthogonal walls. It is worth mentioning that corner collapse due to high thrusts from the roof, analysed in section 3.1.3.3, is a different failure than corner damage due to lack of connections between orthogonal walls. From the experience of past surveys, it is known the importance of fully controlling the corners and connections between horizontal and vertical elements.

The presence of openings always indicates a potential vulnerability of the building. Too many openings or openings with oversized dimensions lead to excessively slender piers in the masonry wall, which is a weak feature in case of an earthquake. Crack lines follow the distribution of the façade openings, proving the vulnerability induced by these elements. The presence of openings facilitates the activation of in plane mechanisms due to the reduction of the load bearing system.

In Norcia, an opening, figure f), very close to the corner motivated the manifestation of cracking. As the corners are an area of high stresses in the presence of seismic actions, openings should not be close to it to not decrease the resistant area of the load bearing walls. For this reason, walls openings should be regular and minimized to improve earthquake resistance to the masonry building.

a) 

b) 

c) 

d) 

e) 

f) 

Figure 5.12 -Corner failures due to lack of confinement in pictures a), b), c), d) and e).
Picture f) is the result of an opening too close to the corner

5.1.5. LACK OF SHEAR CAPACITY OF THE MASONRY WALLS

Damage to stone masonry walls due to in-plane seismic effects, in the direction of the wall length, is less common than damage due to out-of-plane seismic effects. Earthquake loads increase the shear force and can damage the masonry walls and their connections. When subjected to in-plane earthquake loads, masonry walls can demonstrate rocking and diagonal cracking (figure 5.13) or sliding shear, section 3.1.2. Several factors influence the in-plane failure mechanism of stone masonry buildings, including piers dimensions, wall thickness, openings sizes, building height, and masonry shear strength.

In figure 5.14 the pictures a) and b) represent X-shaped cracks only in the pier panels while pictures c) and e) also exhibit cracking in the spandrel panels. Disintegration of the large pieces of the plaster and detachment of the corner accompanies the diagonal cracking in the picture c). Figure d) exhibits a diagonal crack in an electric tower due to shear forces.

In the case of the picture f) failure was caused by rocking of the piers due to the translation of the roof, which resulted in the crushing of the pier end zones.

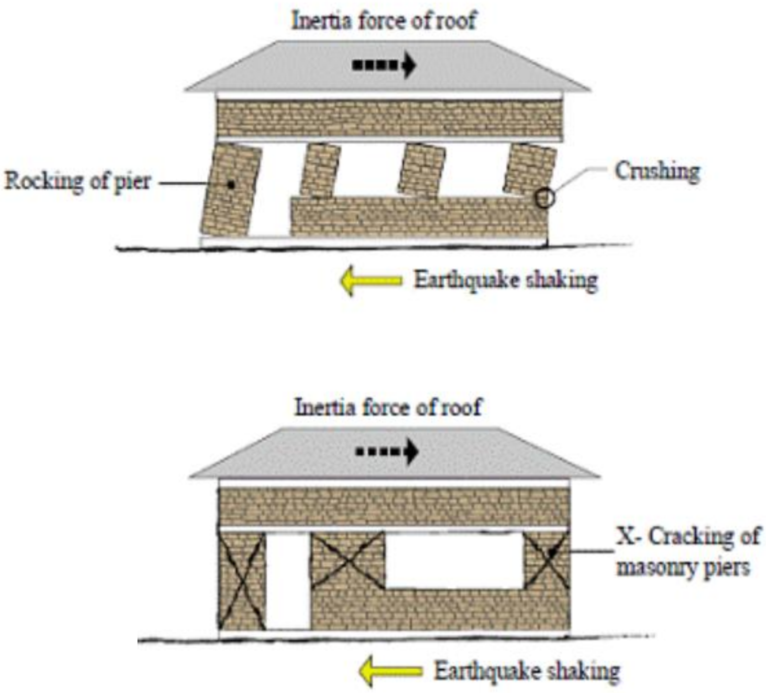


Figure 5.13 -In-plane damage of stone masonry walls: on top rocking failure, and on the bottom diagonal shear cracking (adapted from: Murty 2005)

a) 

b) 

c) 

d) 

e) 

f) 

Figure 5.14 -In plane shear failure in both pier and spandrel panels and f) translation of the roof causing the rotation of the pier

5.2. Most Frequent Deficiencies in Reinforced Concrete Buildings

Reinforced concrete buildings behaved well during the 24th August when they were under the regulations of the seismic code. The ones which were not suffered severe damage, especially related to non-structural elements. Regarding the structural elements, crisis in the beam-column joint, soft stories and columns shear failures were reported.

Main causes for the damage were lack of stirrups in the columns and in the beam-column joints or excessive spacing between stirrups, use of "smooth bars" instead of high-adherence bars in the columns and detrimental interaction between the reinforced concrete frame and the masonry infill walls.

In the section 3.2.3 the failures by lack of stirrups/too spaced stirrups were already detailed and examples of Amatrice and Pescara del Tronto were showed.

5.2.1. Use of "Smooth Bars"

Both columns in figure present "smooth bars" which prevent a good interlocking between the reinforcing bars and the surrounding concrete and facilitated the expulsion of the concrete, section 3.2.4. Also, the excessive spacing of stirrups prevented a good confinement in the columns, leading to the buckling of the transverse reinforcement, figure a).

Figure 5.15 -Wrong type of reinforcement in the columns, Pescara del Tronto. In c) is shown the detail of the smooth bars in a reinforced building collapsed

5.2.2. Detrimental Interaction Between Infills and Frame

When ductile, reinforced concrete frames are designed to resist large displacements without collapse, as it is the case of earthquakes. Masonry infills should be isolated from the confining frame by sufficient gaps at the top and on both sides. The isolation, gap, between the infill and the frame must be greater than any possible deformation expected by the frame, to prevent any infill/frame interaction. In this way, masonry walls do not affect the frame performance and frame displacements are not restrained.

5.2.2.1. In Plane Failure

During earthquakes, masonry infills walls are subjected to high in plane shear forces because of their high initial stiffness. Tension cracks are formed along the loaded diagonal in infill walls, which causes reduction in their lateral strength. In addition, the connection between the reinforced concrete frame and the masonry infill walls is generally weak and infills may get separated from reinforced concrete frames during the earthquake motion, and therefore become susceptible for collapse in the out of plane direction. In the case of figure 5.16 d) the infills walls had a negative shear effect on the column, pushing it outwards and putting in risk the stability of the whole building. The picture c) demonstrates one building which behaved well during the seismic event only suffering in plane and out of plane failures of the coatings. In pictures, a) and b), the separation between frame and infills is accompanied by expulsion of material in the corners.

Figure 5.16 -In plane failures of masonry infills of reinforced concrete buildings in Pescara del Tronto and Amatrice

5.2.2.2.Out of Plane Failure

Once masonry walls crack, continued shaking can easily cause the collapse of infills walls, which is a big danger to the inhabitants. Within this failure, it can both occur collapse of the entire infill wall or just the external leaf, figure 5.17.

Figure 5.17 -Out of plane of infills walls in Pescara del Tronto and Amatrice

5.3. BEFORE AND AFTER

This sub-chapter aims at showing the buildings as they were before and how they became after the devastating action of the earthquake. Some brief comments on the visible damage are accomplished. The sub-chapters are organized by village, Amatrice, Pescara del Tronto and Arquata del Tronto, and by type of construction, masonry or reinforced concrete.

5.3.1. AMATRICE

a)

One square totally destroyed in the southeast, the most affected area of Amatrice.

b)

Main street in Amatrice, Corso Umberto I, ruined after the earthquake.

c)

Row of buildings devastated in the red zone.

d)

Full collapse of the adjacent building on the right and partial collapse of the other on the left. The part of the building which remanined standing exhibits in plane failure in the pier panels and a vertical crack along the intersection between orthogonal walls (type 3 and type 7 from AeDES classification).

e)

Full collapse of the front façade and interaction between reinforced concrete ring beam and poor quality of the masonry.

5.3.2. Pescara del Tronto

a)

Global collapse of two buildings on the right, and partial collapse of the façade of the white building. The building on the left suffered partial overturning of the upper part of the wall. These two-storey buildings coincide with the typology of structures more damaged within the affected areas: low masonry buildings, section 2.6.

b)

Partial collapse of the upper floor, close to the connection with the roof. Possibly the thrusting roof activated the out of plane mechanism. Evident poor quality of the masonry.

c)

Global collapse of a brick masonry structure. This is one of the many cases where the terrain conditions are very hostile due to its steep slope.

e)

Partial collapse of one masonry building in a steep terrain. In section 5.1.1 is visible the wrong type of masonry used in this structure leading to its failure.

f)

Evident different behaviour between two adjacent buildings: global collapse of masonry buildings on the right and standing building on the left, only displaying in plane cracks. Collapsed building presents absent connection between diaphragm and vertical elements, visible on the right side (red arrow). Regarding the in plane cracks in the left structure, there is a mixture of shear and flexure cracking: windows corners exhibit flexural cracks of both pier and spandrel panels (type 1 and 5 from AeDES classification) and also diagonal shear cracking in both pier and spandrel panels (type 2 and 3 from AeDES classification).

g)

Out of plane behaviour of the brick (*mattoni semipieni*) panels, uncovering old openings that have been closed. Probably the defective connection between the bricks and the steel lintels activated the mechanism.

h)

Pescara del Tronto was all but destroyed after the devastating earthquake.

5.3.3. ARQUATA DEL TRONTO

5.3.3.1. After the 24th August

a)

Partial collapse of a masonry building. Visible poor quality of the masonry and in plane shear cracking in both pier and spandrel panels (type 2 and3 from AeDES classification) with detachment of material in the spandrel panel.

b)

Landslide due to failure of a retaining wall, more detailed in section 4.1.2.1.

c)

Global collapse of the two buildings in the end of the street. Diagonal shear cracks in pier panels (type 3) and overturning of the external leaf of the yellow building, detailed in section 5.1.1.1.

d)

The two adjacent buildings showed completely different behaviours. The one on the right behaved well only exhibiting in plane cracking with detachment of material in the ground floor. The vertical crack along the intersection (type 7 from AeDES classification) with the adjacent building, seems a sign of an incipient out of plane mechanism, simple overturning. The building on the left partially collapsed due to the interaction between reinforced concrete slabs and poor quality of masonry.

e)

Most of the buildings in this area survived the earthquake. From the image it is possible to observe severe damage in two buildings.

f)

Global collapse of a masonry building with concrete roof in the main square of Arquata del Tronto. The adjacent buildings, the pink one, exhibits diagonal shear cracking in the pier panel (type 3 from AeDES classification).

g)

Overturning of the front façade involving three floors due to interaction between two buildings with different stiffness. Also, defective connections between diaphragm and vertical elements in the intermediate floors may have caused the vertical bending mechanism (red arrow in the detail picture).

Pounding is also evident between the lower building and its adjacent. The chimney suffered rotation and sliding, figure 5.19.

With a closer look, it is detectable that the two small windows in the before picture were closed and behaved well during the earthquake event, meaning that its closure was correctly conceived (the original opening must be fully filled to avoid its disintegration under dynamic loads)

The next figure is an example of incorrect closure of an opening in a church located in Norcia uncovered during the 30th October earthquake. This large opening had been hidden for decades. In this case, the connection between the stone arch and the masonry was difficult to achieve.

Figure 5.18- Church in Norcia with uncovered opening due to dynamic loads

Figure 5.19-Damage to non-structural element

5.3.3.2. After the 30th October 2016

a)

Severe damage to the masonry buildings, including partial collapses. Most of the buildings that resisted the August event perished in the following month, as it was expected.

b)

Full/ partial collapse of masonry buildings.

c)

Photo taken from the road, showing the total destruction of the village.

d)

Panoramic view showing the full destruction of Arquata del Tronto.

6
RETROFIT MEASURES

Absence of reinforcement elements in masonry buildings allied to earthquake motions usually is a fatal combination. In situations where these measures are well applied, human lives are saved but there is not much that can be done to improve the seismic performance of some heritage masonry beyond a certain point. However, retrofitting can also compromise the integrity of the entire structure if there is an insufficient knowledge of the bearing system, repair techniques and material properties.

Seismic interventions intend to increase the structural performance of the buildings under dynamic actions by reducing horizontal diaphragm deformability and improving the masonry strength in key points. Nevertheless, in several situations aggressive alterations are performed. One of these types of improving methodologies is the replacement of wooden floors by reinforced concrete, already approached in section 5.1.3. In most of cases, is not simple to classify the retrofitting in good or bad, because the lack of compatibility between the old structure and the new devices allied to the poor workmanship also plays an important role. For this reason, one fundamental aspect to have in mind when it comes to strengthening buildings is the similar mechanical and chemical/physical properties of the new materials in relation to the original ones. The first step to accomplish a satisfactory seismic behaviour, the so-called box behaviour, is to perfectly control the connections between walls/ floor and walls/roof. Over the years many rules and principles have been applied to masonry buildings to avoid failure mechanisms such as quoins, tie rods, buttresses and ring beams. However, as the earthquake has a long return period, these measures were rarely introduced in the masonry construction and when introduced it was after the occurrence of an earthquake in forms of careless and wrong retrofit.

Concerning the earthquake in August 24, the destruction was amplified by vulnerable buildings whose upgrade to anti-seismic codes did not exist. As Italy has been struck by strong earthquakes since centuries, the lack of retrofit elements seems surprising. However, reasons pointed out for this fact were the high costs of implementing the anti-seismic measures, the difficulty of the population of getting finance and the modifications were too complex to get approval. The costs of evaluating and retrofitting old structures were an obstacle also because many of the houses were summer houses and were not a primary residence worth a considerable investment. Despite these facts, some experts on seismic engineering and seismology still believe Italy is among the countries with best anti-seismic standards. The current norms are only applied to new buildings and so, for the majority of the buildings there are no anti-seismic classification and it is difficult to understand whether one building is dangerous or not during an earthquake.

One of the polemic cases was the school Romolo Capranica which was renovated in 2012 and supposed to resist powerful earthquakes. However, it was left in ruins, showing the danger of the negative retro-fitting. The past holds similar events: in L'aquila 2009, the university dormitory collapsed and in Molise 2002 the collapse of an elementary school killed 26 children.

Figure 6.1 represents a passageway common in historic centres, in this case in Arquata del Tronto, that also acts as a bracing element between the row of buildings under the seismic action. For this reason, it could be included as a passive measure against earthquakes.

The next sub-chapters will detail the reinforcement elements found in old masonry buildings in Amatrice and nearby villages such as quoins, buttresses, ties rods, ring beams and seismic joints.

Figure 6.1-Passegeway acts as bracing element under earthquake action, Arquata del Tronto

6.1. Stone quoins and buttresses

6.1.1. Stone Quoins

Quoins consist in large squared stone blocks used at the corners to improve the 'box behaviour' of the building and preventing the walls to overturn by providing an effective corner confinement. Unconfined corners, as seen in section 5.1.4, were one of the main causes of damaged masonry buildings. However, when the connection between the orthogonal walls is poor, the efficacy of the quoins is limited and masonry tends to become loose. The following figure is from buildings in Amatrice centre which be-haved well under the seismic action

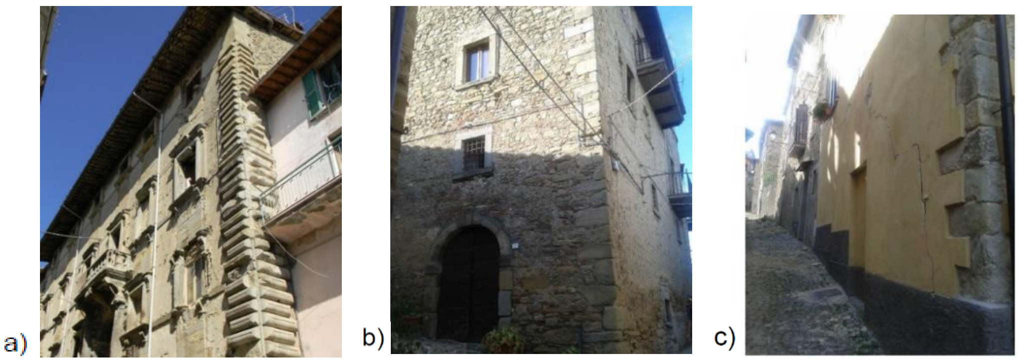


Figure 6.2-Insertion of quoins in the corners of buildings, Amatrice. The picture c) also demonstrates lack of quality of the masonry, as it is visible the stair stepped path in the wall

6.1.2. Buttresses

Over the centuries, buttresses have been used to stabilize and improve the structural capacity of the masonry from the exterior. Basically, it consists in a mass of masonry built against the walls, figure 6.3. Generally, this measure is accompanied by the widening of the walls at the base of the façades. In the figure a) is visible that the buttress stopped the diagonal crack that was developing in the corner of the building, being this one successful case of corner reinforcement. The figure c) on the right, the buttresses were collocated by floors, a different variant from the previous case but, in this situation both in plan and out of plane mechanisms were not prevented.

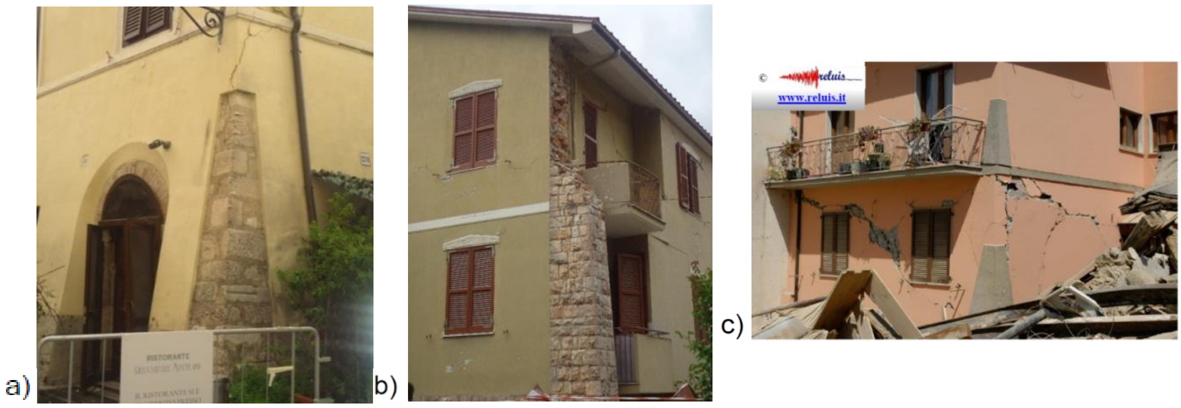


Figure 6.3-Buttress a) and b) in Norcia and c) in Amatrice In a) the reinforcement prevented the spreading of cracks. c) severe in plane and out of plane damages

6.2. Tie Rods

Out of plane overturning of the wall façades is the most frequent failure mechanism, in the earthquake of 24th August and in all the surveyed past earthquakes in Italy. As this mechanism is strictly linked to the poor connections between horizontal and vertical elements, the use of ties in masonry buildings is a very important factor of effectiveness of the restrains. This subject is well documented in literature and has been studied by several authors. Figure 6.4 compares the behaviour of two buildings one with ties b) and the other without this retrofit measure a): in the first case, the out of plane collapse is expected whereas in the second case the structure can behave like a box, only exhibiting in plane failures.

Tie rods usually are located at the floors and/or roof level along the two main directions of the building, anchored by metallic plates of different shapes (figures 6.7 a) and c), rose head shape and wrought steel cross tie introduced in stone quoin, respectively). The bigger the plate is the more resistance is offered against the seismic action. For an efficient application, tie rods should have a feasible stiffness, including a big diameter and reasonable length, and the pre-stressed force must be precisely controlled so the range of dimensioned values is attained. Also, the interface between metallic plates and tie rods must be compatible with the structural capacity of the wall masonry in order to prevent local failures. Never should the anchorage be located within the wall section in cases of poor quality of the masonry. Finally, it is very important to accomplish a regular distribution of ties, as it is shown in the figure 6.4 and masonry should have quality.

The most effective arrangement is a pair of twin chains placed parallel along the same wall, figures 6.5 a) and 6.7 d). Regarding the wrought steel tie rods, they should never be placed vertically or horizontally: the ideal orientation of the plates is 45° with the vertical line with the upper arm facing the masonry wall and the lower arm facing the floor slab (figure 6.6) so, the tie can discharge the seismic forces in both directions. However, in the past this technique never became part of the current construction, figure 6.8 c) and d).

Figure 6.7 exhibits buildings representative of good performances, with tie rods. In case a), the existence of the reinforcement element possibly stopped the corner disaggregation, developed due to the poor quality of the masonry. In case b), the tie rod seems to have prevented the expansion of diagonal cracks and maybe also have avoided the out of plane failure of the plaster. The others do not show any damage. In figure d) the tie rods seem to have avoided the out of plane mechanism.

Figure 6.8 exhibits the opposite situation, where the tie rods were not capable of preventing the out of plane failure and the walls overturned. In case b), the tie rod seems wrongly located inside the slab, which limits its action. Picture c) represents the collapse of one church in Amatrice and it is visible the bad position of the tie rods: they are located at half of the wall, instead of close to the connection between the roof and the walls, and are vertically aligned. In the case d) the horizontal arch mechanism occurred due to the lack of one tie in the centre of the façade. Also, the opening too close to the corner of the building facilitated the failure, by reducing the load bearing capacity of the wall.

a) b)

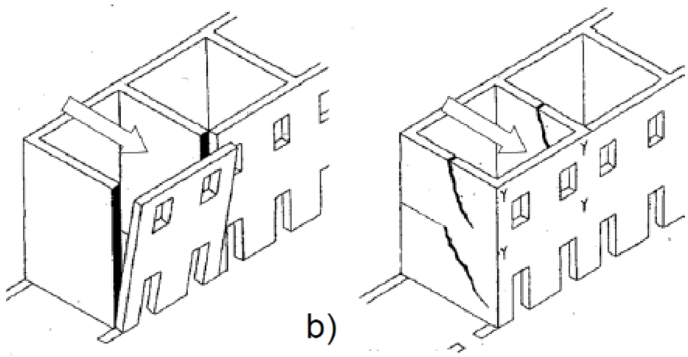


Figure 6.4 -Different mechanisms due to the presence of tie rods: a) out of plane and b) in plane

a) b)

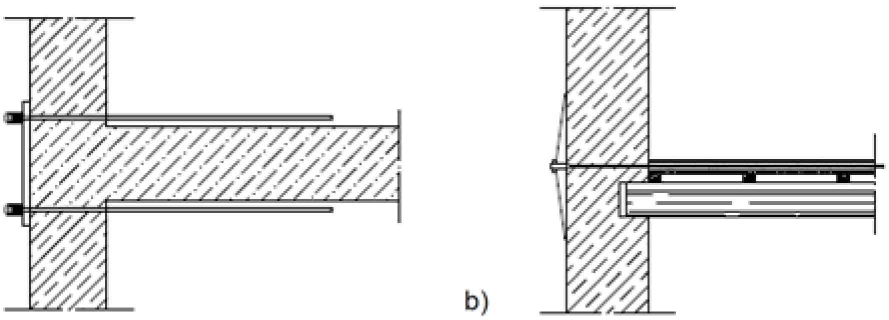


Figure 6.5 -a) Plant view of the twin chains adjacent to the walls at the slabs level. b) Lateral view of simple chains

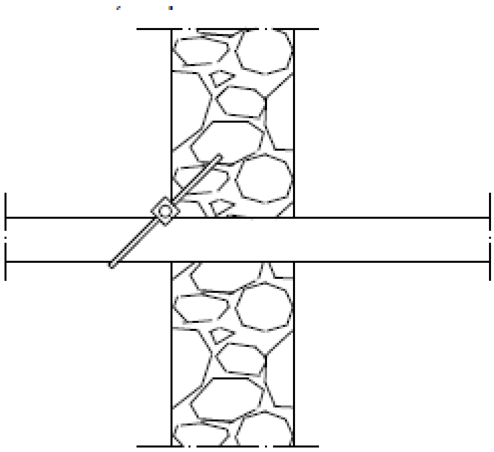


Figure 6.6 -Right position of the tie rod in a masonry building

a) 

b) 

c) 

d) 

Figure 6.7-Good performance of tie rods,a) Pescara del Tronto, b) and d) Arquata del tronto, c) Norcia

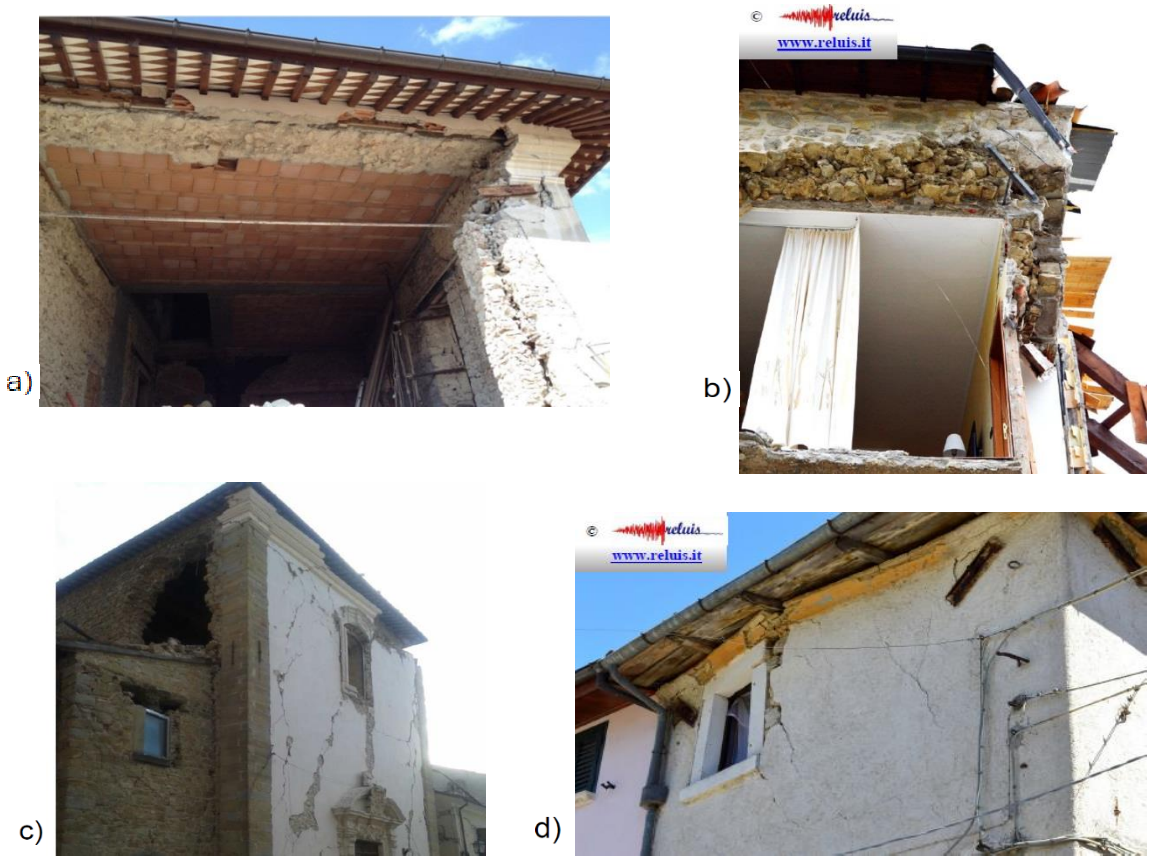


Figure 6.8-Failure of masonry buildings with tie rods. a), c) and d) Amatrice and b) Arquata del Tronto

6.3. EXTERNAL CONFINING STEEL CABLES

In the hamlet of Borgo, near Arquata del Tronto, there were found cases of confinement through external steel cables, figure 6.9. This type of reinforcement also aims at improving the global behaviour of the buildings by strengthening the connections between horizontal and vertical elements.

Figure 6.9 -Through-going iron bars, Borgo

6.4 RING BEAMS

A reinforced concrete steel ring beam constructed at the roof level is one of the most effective measures to prevent the out-of-plane collapse of masonry walls. Displacements of the roof structure are prevented by anchoring its elements into the ring beam. The ring beam contributes to the reduction of the out-of-plane stresses on the upper part of the roof supporting wall but, it does not retrofit the roof to resist additional vertical loading. The connection between the ring beams and the masonry walls is made through steel dowels and anchors, figure 6.10. This is an example of a ring beam however other variants are frequent. The fundamental aspect is to achieve an effective connection between the new reinforced concrete ring beam and the existing masonry, otherwise the earthquake causes severe damage. The main disadvantage of this intervention is that a very large mass is placed at the top of the structure, and in some cases this clearly contributed to overall failure.

However, it is hopeless to introduce retrofit instruments if the masonry quality is still very poor and incapable of resist the earthquake action, which occurred in many cases in the affected area and figure 6.11 shows some examples. In case c) it is visible the anchors, probably due to a bad connection between the reinforced concrete ring beam and the poor quality masonry wall.

In other situations, the ring beams were introduced at the floors level, figure 6.12. The ideal situation would be to have both roof ring beam and floor ring beam in the structure but, at the time of the construction of Amatrice and nearby villages, this practise was not applied. The problem of picture b) is the bad positioning of the steel ring beam within the masonry wall. If the steel ring beam is not all in contact with the masonry wall its action is limited and the out of plane mechanism is allowed. In figure a) the steel ring beam also seems in a bad positioning within the wall, and it cracked due to the shear interaction with the partition wall.

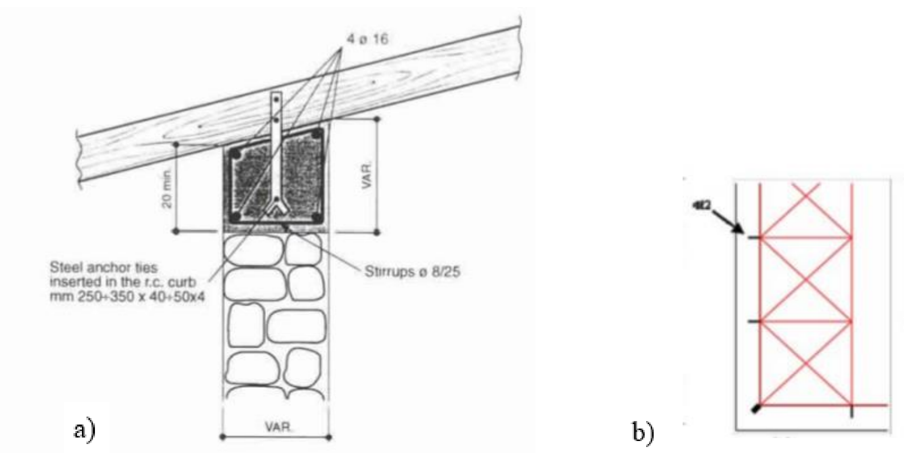


Figure 6.10 -a) section view of a ring beam at the roof level. b) plant view of the ring beam

Figure 6.11-Interaction between reinforced ring beams in poor quality masonry, Amatrice

Figure 6.12 -Bad positioning of steel ring beams at the floors level, Accumoli

6.5. Seismic Joint

Seismic joints are used to divide a complexly shaped building into a group of smaller buildings with simple shapes that are easier to study and have a predictable seismic behaviour. In cases of L-shaped buildings the division is often made into two rectangles. They are also used in cases of building constructions adjacent to existent ones. The action of seismic joints during the earthquake is, in short words, to accommodate the seismic forces in both horizontal and vertical directions.

The following figures present the school Romolo Capranica in Amatrice. More than 700000 euros were spent on the improvement of the school and one the measures was the introduction of the seismic joint. Despite this reinforcement, 5 years after the school collapsed. As mentioned before, this building was constituted by blocks of masonry and blocks of reinforced concrete, figure 6.14. The seismic joint in figure 6.13 avoided the collapse of the reinforced concrete part but allowed the failure of the masonry block. This could be explained based on the poor masonry quality and poor constructions techniques, similarly to the majority of the old masonry in the region.

Figure 6.13 - Collapse of the masonry block and visible seismic joint, school of Amatrice

Figure 6.14- Pre-event image where the two different blocks are well recognisable (masonry and reinforced concrete blocks)

7

CONCLUSIONS

The death toll and destruction caused by the 24th August, 2016 M 6.0 earthquake that hit central Italy, reminds once again how fragile and unprepared the Italian territory is to withstand seismic actions. Seismic history is consistent with this kind of event and holds past earthquakes much more devastating in the same region. These continuous seismic actions are the result of a series of active faults in the Apennines chain and, for this reason, Amatrice and nearby villages are located in the zone with more seismic risk of Italy. Amatrice and Pescara del Tronto were the furthermost affected villages.

Despite its moderate magnitude, the earthquake resulted in tremendous damage. This can be linked to the very shallow depth normal fault rupture (8 km), poor construction techniques and/or poor quality of materials used in masonry buildings (the historic centres of the towns destroyed by earthquake had ancient centuries old constructions) and sites effects such as topographic amplification, which can be expected in case of towns located on top of ridges. In terms of site effects, the GEER found signs of site amplification in Arquata del Tronto which, allied to the buildings vulnerability (fatal combination), caused the major destruction. In Amatrice and in Pescara del Tronto no clear patterns of damage suggesting site effects were detected but a deep investigation is needed to ensure these statements. Another fact which contributed to the widespread destruction was related to the acceleration response spectrums recorded by the four stations with less epicentral distance. These spectrums showed that the earthquake excited mainly short periods coincident with the periods of masonry buildings.

Regarding the damages to the three affected villages, Amatrice, Pescara del Tronto and Arquata del Tronto, it can be included the damage to residential, public and historical/cultural buildings and the occurrence of dozens of landslides and rock falls due to the steep terrains. The building stock in the region was constituted mainly by old unreinforced concrete buildings with generally two floors in historic centres and reinforced concrete buildings, mostly four-stories, in the suburbs. Damages to historical/cultural were generalized among the hamlets and many collapses of churches were detected. Concerning the public buildings, in Amatrice were reported severe damages to the police station, school (retrofitted five years before) and hospital, demonstrating once again the urgent need to improve correctly the masonry built. Damages detected to reinforced concrete buildings were mainly:

- Severe failures to non-structural elements: widespread cracking in external infills and out of plane failure of infills panels due to detrimental interaction between infills and frame;
- Damage to structural elements: brittle failures of columns and crisis of the beam-column joint due to lack of adequate seismic design and use of “smooth bars”;
- Brittle failures caused by soft storey mechanism.

Damages detected to masonry buildings were the following:

- Out of plane failure of the walls due to absent diaphragms and/or ineffective connections between diaphragm and orthogonal walls;
- Collapse of buildings and out of plane failure of walls due to poor quality and heterogeneity of the masonry;
- Collapse of floors due to ineffective connections between diaphragm and orthogonal walls;
- In plane shear failure in both pier and spandrel panels, mainly X-shaped cracking, with detachment of the plaster in some cases;
- X-shaped cracking or vertical cracking in the buildings corners due to an absent confinement between orthogonal walls;
- Full collapse of buildings with reinforced concrete roofs;
- Hammering of masonry walls by reinforced concrete roofs.

Appropriate measures of seismic prevention are particularly urgent in these high seismic risk regions to reduce the high vulnerability of built heritage. When the retrofitting is accomplished without a sufficient knowledge of the bearing system and repair techniques the result is the increase of building vulnerability. In the region, most of the masonry buildings were not improved mainly because of the costs associated with the process and also because the majority of the buildings were summer houses. Moreover, the current norms only apply to new buildings and for this reason the law is dubious in terms of old masonry buildings. In addition to this, many times the lack of compatibility between the original old masonry and the modern retrofit techniques was an impediment to implement the seismic retrofit measures. The structures which were improved presented distinct upgrade measures such as ring beams, tie rods, quoins, buttresses, external confining steel cables and seismic joints. In the overall, in some cases these measures produced good results but, in others worsened the building seismic behaviour. Most of the errors detected were the bad positioning of the tie rods within the slabs and their vertical alignment, instead of being orientated 45° with the vertical line. In other cases, they were too spaced and allowed out of plane failures. Regarding the buildings improved with ring beams, most of the failures detected were due to the poor quality of the masonry and/or bad connection between the masonry wall and the ring beam. When the ring beams were introduced at the floors level was detected a bad positioning of the steel ring beam within the masonry wall.

Simply based on past history, in the next decades more earthquakes will affect the Italian territory. It is not possible to guess when, but it is certainly in the areas where capable active faults are mapped. An urgent need to rebuild well and safe, with widespread adoption of adequate and effective mitigation measures is obvious. Regarding the new constructions, the design must follow the seismic norms. Only with the implementation of these rules future disasters like the August 24, 2016 will be avoided.

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